

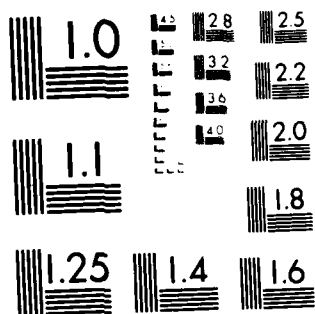
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SECTION 32 PROGRAM
STREAMBANK EROSION CONTROL
EVALUATION AND DEMONSTRATION
WORK UNIT 2-EVALUATION OF EXISTING
BANK PROTECTION

FIELD INSPECTION OF SITES IN THE
MISSOURI RIVER DIVISION

by

Malcolm P. Keown, Elba A. Dardeau, Jr., Etta M. Causey

Hydraulics Laboratory

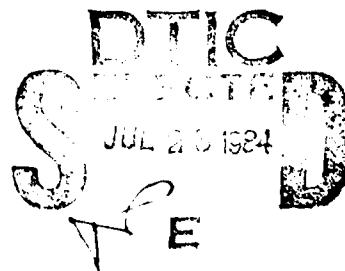
U. S. Army Engineer Waterways Experiment Station
P. O. Box 631, Vicksburg, Miss. 39180

October 1980

Inspection Report 10

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SECTION 32 PROGRAM
STREAMBANK EROSION CONTROL EVALUATION AND DEMONSTRATION
WORK UNIT 2 - EVALUATION OF EXISTING BANK PROTECTION

FIELD INSPECTION OF SITES IN THE MISSOURI RIVER DIVISION

1. Section 32 Program existing sites* in the Missouri River Division (MRD) were inspected 18-21 September 1978 by the U. S. Army Engineer Waterways Experiment Station (WES) evaluation team. These sites were located in the U. S. Army Engineer Districts, Kansas City (MRK), and Omaha (MRO). WES personnel participating in the effort were Messrs. E. B. Pickett and N. R. Oswalt, Hydraulics Laboratory, Dr. E. B. Perry, Geotechnical Laboratory, and Mr. M. P. Keown, Environmental Laboratory (EL). The WES team was accompanied by Messrs. W. J. Mellema (MRD), W. M. Linder (MRK), and J. D. Nelson (MRK) during the inspection effort. This report was written by Mr. Keown, Ms. E. M. Causey (EL), and Mr. E. A. Dardeau, Jr. (EL). Mr. R. M. Russell, Jr. (EL), prepared the figures.

2. The following existing Section 32 Program existing sites were visited in MRD (Figure 1):

- a. Little Blue River at Independence, Mo. (3 sites):
- b. Mud Creek at Lawrence, Kans.
- c. Republican River at Milford Dam, Kans.
- d. Tributary of the Black Vermillion River at Frankfort, Kans.
- e. Big Blue River near Marysville, Kans.
- f. Deadman's Run and Antelope Creek at Lincoln, Nebr.
- g. Floyd River at Sioux City, Iowa.
- h. West Fork Ditch at Onawa, Iowa.
- i. Little Sioux River at Onawa, Iowa.
- j. 102 River at Bedford, Iowa.

During inspection of the Little Blue River sites, Messrs. R. G. Dale (MRK) and J. L. Burton (MRK) accompanied the team. Mr. D. R. Taladay, an engineer with Clark Enerson Partners of Lincoln, Nebr., joined the inspection team during the visit to Deadman's Run and Antelope Creek. Messrs. J. E. Eagleton (MRO) and J. F. Kelley (MRO) participated in the inspection effort during visits to the Floyd River, West Fork Ditch, and Little Sioux River sites. Special acknowledgment is made to Messrs. Linder, Dale, and Eagleton as well as to Messrs. H. D. Wisenborne (MRK), F. Vovk (MRO), J. E. Dover (MRO), and R. L. Young, Clark Enerson Partners, for their able assistance in providing the information needed to prepare this report.

* Existing sites are those locations where the bank protection works were not constructed with Section 32 funds but will be monitored under the Section 32 Program. Protection works at demonstration sites are constructed and monitored under Section 32 funds.

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Little Blue River at Independence, Mo. (Mile 7.4 to 11.5*)

3. The Little Blue River is a right bank tributary of the Missouri River, joining the main stem at mile 339.5 (1960 adjustment) 20 miles downstream from Kansas City, Mo. (Reference 1). The Little Blue Basin is 33 miles long, with a maximum width of 13 miles. The total drainage area of the basin is 224 square miles of which 90 percent is in Jackson County, Mo., with the remainder being in Cass County, Mo. The lower 7.4 miles of the Little Blue River are confined to an improved channel between the right bank bluffs of the Missouri River floodplain and the Unit R-351 tieback levee of Missouri River Levee System.

4. The Little Blue River Basin is frequently subject to flooding from high-intensity rainstorms mostly during the months of April through October. Flood stage at the Lake City gage (Figure 2) has been exceeded 21 of the 23 years since records have been kept. The gradual encroachment of the Kansas City metropolitan complex into the basin has significantly raised the flood damage potential. To mitigate this threat, a project for channel improvement and reservoir construction was authorized for the Little Blue Basin by the 1968 Flood Control Act, Public Law 90-483 (part of the comprehensive plan for the Missouri River Basin). This legislation provides for channel improvement (in four stages) from mile 7.4 through mile 22.4, and the construction of Longview Dam upstream from the main-stem channel improvements and Blue Springs Lake Dam on the East Fork of the Little Blue River. The channel improvement feature incorporates two types of channels: a low-flow channel that follows much of the bed of the existing Little Blue River, and a high-flow channel 5 ft in elevation above the existing waterway to handle flood discharges. When the project is completed, the natural channel length will be shortened from 22 miles to 15 miles for the high-flow channel and to 18 miles for the low-flow channel. The total cost (1978) of the channel improvement feature is \$30,000,000 and for the two dams \$121,000,000.

5. The Stage I channel improvement contract was awarded on 20 December 1975 (Figure 3) and was completed on 5 December 1978 (Reference 2). Three Section 32 existing sites were selected in the Stage I reach:

- a. Stone riprap on the side slopes of the low-flow channel.
- b. Sheet piling and rock sills.
- c. Compacted clay on the berm and side slopes of the high-flow channel.

The structures placed at each of these sites were designed to withstand a 100-year flood. The crest elevations of the sheet piling and rock

* Mileage estimated from 1:24,000 USGS topographic quadrangle maps for Buckner, Mo., Missouri City, Mo., and Blue Springs, Mo., all photo-revised through 1975.

sills through the Stage I reach range from 708.5 to 721.9 ft.*

6. Since March 1948, the U. S. Geological Survey (USGS) has maintained a stream gaging station at the Missouri State Highway 78 Bridge (reported as the Lake City gage; see Figure 3). The daily discharges of record are: Maximum 17,000 cfs (during the September 1977 flood); mean 133 cfs; and minimum no flow on several occasions (Reference 3). The maximum observed stream velocity is 9 ft/sec which occurred during a discharge of 16,100 cfs. A suspended-sediment sample station has been operated at this site by MRK since October 1971; for the period of record the maximum load is 253,144 tons/day, the mean 426 tons/day, and the minimum 0 tons/day (Reference 4). The maximum annual load of record is 374,933 tons (water year 1977); the average annual load is 155,556 tons. The average annual sediment yield upstream from the Stage I reach is 750-1,000 tons per square mile (Reference 5). Soil types vary from clays in the upper end of the Stage I reach (CL and CH) to sands at the lower end (SM, SP, and sandy ML).

7. The side slopes of the low-flow channel were protected by an 18-in.-thick layer of Type A stone riprap placed over a 6-in.-thick layer of bedding material in reaches where minimum protection was required (Figure 4); in high-velocity or turbulent environments (bends, downstream from structures, etc.), a 21-in.-thick layer of Type B riprap was placed over the bedding material. Riprap was required at 12 locations through the Stage I reach (Figures 3 and 5) to stabilize the low-flow channel. All materials were brought onsite by truck; an orange peel bucket and crane was then used for below water placement, and a Gradall for above water placement. The Type A and Type B stone riprap was specified to meet the following gradations:

<u>Weight per Stone</u> <u>lb</u>	<u>Percent of Total Weight</u> <u>Lighter than</u>
<u>Type A</u>	
250	100
180	85-95
60	30-50
10	0-10
<u>Type B</u>	
600	100
450	85-95
150	30-50
20	0-10

The riprap was required to be approximately rectangular in cross section, to be relatively free from slabby pieces and deleterious substances, and to have an elongation ratio not exceeding 3. The bedding material was specified to meet the following gradation:

* All elevations noted in this report are referenced to mean sea level.

<u>Sieve Size</u>	<u>Percent by Weight Passing</u>
4 in.	100
3 in.	75-95
3/4 in.	40-60
3/8 in.	20-40
No. 4	5-25

Material not passing the 3/4-in. sieve was specified to be reasonably free from flat elongated particles and deleterious substances.

8. Shortening of the natural channel through the Stage I reach required placement of five sheet-piling and rock-sill structures* to maintain bed-gradient equilibrium (3 ft/mile; see Figures 3, 6, and 7). The sheet piling conformed to Military Specification MIL-P-11858, Type II, Section Z-27 (Reference 1). The interlocking piling was driven with a pile hammer and then trimmed (Figure 8). The remainder of the structure consisted of Type D riprap placed on a 12-in.-thick layer of spalls material, which in turn rested on a 6-in.-thick layer of bedding material (Figure 7).

9. The Type D riprap was required to meet the following gradation standards:

<u>Weight per Stone</u> <u>lb</u>	<u>Percent of Total Weight</u> <u>Lighter than</u>
2,000	100
1,500	85-95
500	30-50
70	0-15

The 12-in.-thick layer of spalls material was specified to meet the following requirements:

<u>Sieve Size</u>	<u>Percent by Weight Passing</u>
12 in.	100
8 in.	75-95
4 in.	40-60
1/2 in.	5-25

As with the Type D riprap, the spalls were required to be approximately rectangular in cross section, to be relatively free from thin slabby pieces, and to have an elongation ratio greater than 3. The 6-in.-thick layer of bedding material placed under the spalls was required to meet the same criteria as the bedding material used for the low-flow channel side-slope revetment (paragraph 7). The in-place cost of the sheet piling and rock sills was not separable from the total project cost.

10. Preproject boring logs indicated that much of the proposed

* A sheet-piling and rock sill structure was in place at sta 737+00 prior to the Stage I channel improvements.

high-flow channel side-slope and berm surfaces in the lower 3 miles of the project would be of noncohesive material if in situ materials were used. To minimize bank erosion in this reach, the sand was replaced by an impervious clay blanket. The blanket material was specified to be of low permeability, consisting of clays (CH) and (CL), and to be free of plant growth, roots, and humus. The composition of the impervious material was such that a minimum of 50 percent of the soil particles by weight must pass a U. S. Standard No. 200 sieve. The minimum liquid limit of the material was specified to be 40. After bank preparation, the material was placed in one lift to a final minimum thickness of 12 in. (Figure 9). Compaction was accomplished by two passes of a crawler tractor. To assure channel stability, the final project design required that 11,547 lin ft of the high-flow channel side slopes and berms be covered by a clay blanket from sta 671+00 to 784+00, representing 51 percent of the total length of the banks between these stations (Figure 3). In addition, a 700-ft segment of the right bank below the Missouri State 78 Highway Bridge was blanketed. The impervious blanket placement required 26,553 cu yd of material at an in-place cost of \$2.00/cu yd (1975). The total area covered was 16.5 acres.

11. The clay blankets were seeded with a grass mixture as follows:

<u>Seed Type</u>	<u>lb/acre</u>
Tall Fescue	15
Domestic Rye	6
Smooth Brome	15
Reed Canary	10
Total	46

After the seeding was completed, mulch was placed over the seeded areas. The bidcost (1974) for materials (seed and mulch) and the planting operation was \$825/acre.

12. The flood of record in September 1977 (17,000 cfs) caused no damage to any of the structures in the Stage I reach. Collection of excessive debris has occurred behind some of the sheet piling and rock sills (Figure 10). MRK personnel report that as a result of uncontrolled overbank drainage, clay blanket erosion has occurred at some locations (Figure 11). At the time of the WES inspection visit, the overall project was in an as-built condition (Figure 12). MRK visits to the Stage I reach through January 1980 indicate that the bank protection measures are performing well; the only problem noted has been surface erosion of the clay blankets.

Mud Creek at Lawrence, Kans. (Mile 0.8 to 2.0)

13. Mud Creek is a left-bank tributary of the Kansas River joining the main stem at Lawrence, Kans. (Kansas River mile 46.8). Streamflows in the Mud Creek Basin originate 14 miles due north of Lawrence and move

in a southerly direction to a point near Midland, Kans., about 4 miles north of Lawrence. The stream then flows in a southeasterly direction until it converges with the Kansas River. The total stream length is 20.4 miles, with the gradient of the channel being approximately 6.5 ft/mile (Reference 6). The Mud Creek watershed has a long rectangular shape with a total area of 38 square miles. The watershed upstream from Midland has an area of 30 square miles with topography typical of a small midwestern basin, i.e., a relatively narrow floodplain and steep side slopes. Downstream from Midland, the Kansas River has meandered through the area, apparently eroding away any natural hills along the right bank of the Mud Creek Basin, thus leaving hills adjacent only to the left bank.

14. During the relatively frequent low discharge floods (less than 4,500 cfs) that occurred before channel improvements by MRK, the Mud Creek Basin could contain these floodwaters within its boundary; however, during flood events of 4,500 cfs or greater, excess discharges flowed into the Kansas River floodplain. Although there were a few natural levees and a number of low-elevation artificial levees (constructed by local residents), these topographic features and structures were easily breached or outflanked by the high discharges. Two of the largest storms were in June 1966 and June 1967. MRK computed that the 1966 flood flow peaked at 8,800 cfs on 13 June based on a unit hydrograph and between 8,000 and 10,000 cfs using high-water marks. Flood and hydrographic data were not collected during the 1967 storm, but local residents estimated that stages were equal to or greater than those of 1966 (References 6,7).

15. There have been no stream gaging records maintained within the Mud Creek Basin; thus hydrologic computations have had to be made synthetically based on flood marks and on records of stream gaging stations located near the basin. No suspended-sediment sample collection stations have been operated in the basin. Estimates indicate that average annual sediment yields over the drainage are 1,000-3,000 tons per square mile (Reference 5). The surface soils along the banks of Mud Creek are predominantly lean clays (CL).

16. Flood-control improvements for the Lawrence, Kans., area were authorized by the 1954 Flood Control Act (Public Law 83-780), as a part of the Missouri River Basin comprehensive plan for flood control. The plan of protection for the Mud Creek Unit consisted of the following major elements (Reference 6):

- a. Placement of levees (23,020 ft).
- b. Channel improvement (including grade control) from the mouth to the upstream limits of levee construction (6.1 miles).
- c. Bridge improvements, including removal of existing bridges and construction of new bridges as required to eliminate potential constrictions.
- d. Drainage structures for removal of interior drainage.
- e. Stone riprap protection.

At the request of officials of the city of Lawrence, the Mud Creek portion of the Lawrence area project was restudied and as a result the

plans were revised. The major changes from the original design were:

- a. The three-fourths standard project flood design discharge at the mouth of Mud Creek was increased from 14,000 to 19,250 cfs, with a design velocity of 9 fps.
- b. Levee and channel improvements were extended upstream an additional 3 miles.
- c. Bridge construction was revised to accommodate the increased design discharge.
- d. Levee elevations were raised to provide a minimum of 3-ft freeboard to protect against the new design discharge.

The Mud Creek channel improvement and levee work was awarded on 13 May 1976 and completed in July 1978. Upstream from the improvements, conditions have remained essentially the same as the preproject downstream conditions with the channel heavily choked with timber and underbrush.

17. An essential feature of the Mud Creek Channel improvement project was the construction of four sheet piling and rock sills at miles 0.80, 1.31, 1.55, and 1.96 to provide grade control and prevent channel degradation (Figure 13). These four structures were collectively selected as a Section 32 existing site. Plan and cross-sectional views of the sills are provided in Figures 14 and 15, respectively. The completed structures were placed at an approximate cost of \$165,000 (1978).

18. The steel-sheet piling was required to conform to Military Specification MIL-P-11858, Type II, Section Number Z-27. The sheet piles were interlocking (Figure 16) throughout their entire lengths to the drive depth indicated. The riprap portion of the sheet-piling and rock sill structures were placed on a 6-in.-thick layer of bedding material and 12-in.-thick layer of spalls (Figure 15). Type C riprap (42 in.) was used in the center of the channel and Type A riprap (18 in.) on the side slopes. The riprap, spalls, and bedding materials were all placed with a dragline.

19. Stone for the riprap was required to be sound, durable limestone free from cracks, seams, shale partings, and overburden spoil. The riprap was specified to be approximately rectangular in cross section and relatively free from flat and elongated pieces. The quantity of stone having an elongation ratio greater than 3 could not exceed 5 percent by weight. The riprap was graded subject to the following limits:

Weight per Stone lb	Percent of Total Weight Lighter than
<u>Type A (18 in.)</u>	
300	100
200	80-95
80	30-50
10	0-10

(Continued)

<u>Weight per Stone</u> <u>lb</u>	<u>Percent of Total Weight</u> <u>Lighter than</u>
<u>Type C (42 in.)</u>	
4,000	100
3,000	80-95
1,000	30-50
200	0-20

20. Material for the spalls layer was required to be of tough, durable particles. The total of objectionable material, friable particles, and other foreign matter could not exceed 5 percent by weight. The gradation specified for the spalls was:

<u>Sieve Size</u>	<u>Percent by Weight Passing</u>
10 in.	100
7 in.	75-95
5 in.	40-60
3 in.	20-40
2 in.	0-20

The gradation required for the bedding material was:

<u>Sieve Size</u>	<u>Percent by Weight Passing</u>
6 in.	100
4 in.	75-95
1 in.	40-60
3/8 in.	15-35
No. 4	0-15

21. At the time of the WES inspection visit the project was performing as designed (Figures 17 and 18). MRK inspections through November 1979 indicate that the structures are intact and no headcutting is evident through the project reach.

Republican River at Milford Dam Outlet Channel,
Kans. (Mile 6.1 to 7.3)

22. The 1954 Flood Control Act (Public Law 83-780) authorized construction of Milford Dam at mile 7.7 on the Republican River (4 miles northwest of Junction City, Kans.) as a part of the comprehensive plan for flood control in the Missouri River Basin (Figure 19). Milford Dam is a compacted earth and rock-fill embankment with an impervious core, being 6,300 ft long and constructed to an elevation of 1,213 ft (126 ft above the valley floor). MRK began multipurpose reservoir operations on 16 January 1967. The design storage capacity of the reservoir is 1,160,000 acre-ft, which includes 700,000 acre-ft allocated for flood control with the remainder being for multipurpose use.

23. The USGS has operated a gaging station on the Republican River at mile 6.0 since 1 October 1963. Daily discharges of record prior to the date the dam became operational (1 October 1963-16 January 1967) were: maximum 17,200 cfs, mean 675 cfs, and minimum 9.0 cfs. The daily discharges of record after the dam became operational (16 January 1967-present) are: maximum 12,600 cfs, mean 839 cfs, and minimum 15 cfs (Reference 8). No suspended-sediment samples were obtained in the outlet channel reach prior to closure of Milford Dam; however, MRK did operate a suspended-sediment sample collection station at mile 6.0 from 1 October 1967 through 30 September 1974. The daily suspended sediment loads of record were: maximum 11,037 tons, mean 84 tons, and minimum 0.81 ton. The maximum annual suspended-sediment load was 111,172 tons (water year 1974); the average annual load was 31,875 tons.

24. The 8,000-ft-long Milford Dam Outlet Channel was originally excavated through highly erodible fine silts and sands to a 100-ft bottom width with 1V-on-3H side slopes (Reference 9). Riprap was placed on the side slopes for a distance of approximately 1,000 ft downstream of the stilling basin. Within a short period of time the remaining 7,000-ft length of this channel had eroded to a width of approximately 200 ft and was threatening to encroach into a proposed recreation area. To prevent this encroachment, MRK proposed to retain the existing 200-ft bottom width, grade the banks to a 1V-on-3H side slope, and then pave the banks with a 12-in.-thick layer of rock (8- to 24-in. diameter) having a median weight of 25 lb and a maximum weight of 150 lb. The stone was to be placed over a 6-in.-thick filter blanket in order to prevent loss of the fine-grained bank material through the riprap. MRK anticipated that 7 to 10 ft of degradation could occur after the banks had been stabilized. Extending the side-slope revetment to that depth would have required up to 10 ft of underwater excavation. Past experience with these cohesionless soils had shown that 2 to 3 ft was the maximum working depth for excavation without dewatering when placing a controlled thickness of rock. The estimated cost at 1967 price levels of dewatering and placing the rock to the expected depth of degradation was approximately \$900,000.

25. In view of this very high cost, MRK proposed to place a horizontal blanket of rock on the streambed at the base of the side slopes, and thus as the bed degraded and undermined the blanket, the toe would be armored by the downward migration of stone placed as part of the horizontal blanket. The volume of rock in the blanket was initially estimated to be two times the volume required if the slope protection were extended to the expected depth of degradation. The estimated cost using the rock blanket approach was slightly over \$400,000 or less than half the cost of extending the slope protection. Since the amount of rock actually required to provide sufficient protection as the bed degraded was not known, model testing was needed to see if additional savings might be realized.

26. Model testing was conducted by the Mead Hydraulics Laboratory located at the University of Nebraska Field Station near Mead, Nebr. (a facility jointly used by the University of Nebraska and MRD). Ten

different toe geometries were tested. The model tests confirmed that the horizontal blanket proposed as toe protection for the outlet channel revetment would perform as expected when the bed degraded. They also confirmed that the volume of rock in the blanket could provide adequate protection with substantially less than twice the volume required if the revetment was extended to the anticipated depth of degradation.

27. Placement of riprap from mile 6.1 to 7.3 (the Section 32 existing site) began in August 1966 and was completed in early 1969. Based on the results of the Mead Laboratory model tests, the horizontal blankets were placed 3 ft thick with the width varying from 12 to 17 ft. This technique provided a quantity of rock approximately 1-1/2 times the volume that would have been required to extend the slope protection to the expected depth of degradation. Specifications for the 12-in.-thick layer of stone riprap pavement on the side slopes were noted in paragraph 24. The requirements for the 6-in.-thick layer of bedding material beneath the side-slope pavement were:

<u>Sieve Size</u>	<u>Percent by Weight Passing</u>
4 in.	Maximum allowable
2 in.	75-95
1 in.	35-65
1/4 in.	5-30
No. 40	0-15

The completed revetment was designed to sustain a design discharge of 12,500 cfs at a mean velocity of 5 fps (Reference 10). The bed gradient through the Section 32 reach at the time of construction was essentially flat.

28. Four special toe test sections, two on each bank and each 200 ft long, were constructed at the downstream end of the outlet channel (Figure 19). The toe of Test Section 1 was constructed as an extension of the upper bank paving except that the lower 4 ft was placed on a 1V-on-1.5H slope (Figure 20); the base of the toe was 39 in. below the existing streambed at the time of construction. In Test Section 2, the toe was constructed as an enlarged section at the bottom of the revetment (Figure 21); the base of the toe was 16 in. below the streambed. Both the front and back of the toe section were placed on a 1V-on-1.5H slope. The break in slope between the toe section and the 1V-on-3H upper bank slope was located 18 in. above the streambed. The toe of Test Section 3 was constructed as a 12-in.-thick horizontal blanket placed on the streambed (Figure 22); the original width of the blanket was estimated at 18 ft. Test Section 4 was placed on the streambed as a 2-1/2-ft-thick horizontal blanket of rock (Figure 23); the original width of the blanket was estimated at 10 ft.

29. Several periods of prolonged high releases from Milford Lake have occurred since placement of the revetment in the outlet channel. A discharge of 500 cfs or more was passed at least 50 percent of the time between January 1969 and December 1974. For a period of 44 days in 1973,

there were releases of 8,000 to 12,000 cfs. As a result of these flows, the bed of the outlet channel degraded an average of 5 to 6 ft through its entire length.

30. In December 1974, MRK inspected the outlet channel to evaluate the overall performance of the revetment toes. The inspection team found that the toes had performed their intended function well. The team probed the streambed at the base of the slope at several locations to determine the outer extremity of the riprap movement; in addition, they removed a small amount of rock to locate the original elevation of the base of the toe and to estimate the thickness of the stone layer formed by the downward movement of the riprap. They noted that the rock had moved downslope as the bed degraded until the bank was covered with a two-rock-diameter thickness. Approximately half of the toe stone was still in reserve at the original base of the revetment. The four special test sections were inspected in detail in order to compare the performance of the various toe geometries. MRK obtained surveyed cross sections that extended from the top of the revetment to the bed and a short distance out into the channel for each test section (Figures 20-23).

31. Three to 3-1/2 ft of degradation had occurred below the original base of the toe of Test Section 1. The surveyed cross section indicated that the entire riverward face of the toe had receded as the riprap moved downward and adjusted to the degradation (Figure 20). The toe rock had covered the lower slope quite well; however, there was some evidence of separation in the vicinity of the original base of the toe. Since there was little or no stone reserve remaining in this toe, the team concluded that progressive failure of the lower slope protection could occur if there were additional significant degradation.

32. Approximately 4-1/2 ft of degradation below the original base of the toe had occurred in Test Section 2 (Figure 21). Riprap had covered the lower slope to a thickness of approximately two rock diameters. There appeared to be some evidence of separation along the original base elevation of the toe as a line of exposed bedding could be seen along the entire length of the test section.

33. The streambed in the vicinity of the toe of Test Section 3 had degraded approximately 6 ft (Figure 22). The riprap had moved downward to cover the lower slope; however, the thickness averaged somewhat less than one rock diameter, i.e., small areas of exposed sand could be seen over the entire face of the toe. There was a substantial reserve of stone remaining in the toe; however, the MRK inspection team noted that if additional degradation occurred too rapidly, partial revetment failure could occur due to separation of the thin blanket. Alternatively, recession of the bank by leaching could cause additional riprap to move downward from the reserve and form a thicker bank covering.

34. Approximately 5 ft of degradation had occurred along the toe of Test Section 4 (Figure 23). There was a uniform blanket of riprap on the lower slope averaging at least two rock diameters thick. No evidence of blanket separation or areas of exposed bed material were noted. The riverward edge of the stone was essentially at the base of the slope

with very little migration of riprap out into the channel. A substantial reserve of rock that could provide material to accommodate additional degradation still remained.

35. In summary, the 1974 inspection indicated that the overall performance of the prototype test sections confirmed the results observed in the model tests. An extension of the slope protection showed evidence of stress soon after the base of the revetment was undercut (Test Section 1). The enlarged section at the base performed reasonably well; however, it appeared to have a tendency to eventually separate at the point where the steeper slope of the toe intersected the flatter slope of the upper bank paving (Test Section 2). A thin horizontal blanket did not release rock at a rate sufficient to provide an adequate thickness of coverage (Test Section 3). The performance to date of the thicker and narrower horizontal blanket (Test Section 4) was clearly superior to that of the other three test sections.

36. The model tests by Mead Laboratory indicated that only a single layer of riprap would form as the toe rock moved downward and that some channelward movement of stone would occur and form a horizontal apron at the base of the slope. The prototype performance of the thick horizontal blanket showed that a layer several rock diameters thick can develop under actual field conditions. There was also much less movement of riprap channelward at the base of the slope than was indicated by the model tests. Apron formation in the model may have been the result of the relatively large dune pattern in the model bed. In the model tests, the height of these dunes was approximately one fourth of the water depth. At the time of the 1974 inspection, the streambed of the outlet channel was essentially flat, and even during high flows, it was doubtful if dunes of significant height were formed. Final side slopes in the model were approximately 1V on 2H for all toe geometries. In the prototype, the slope below the horizontal toes (Test Sections 3 and 4) was approximately 1V on 2H, however the slope extension (Test Section 1) and the enlarged base (Test Section 2) were somewhat steeper below the original toe (about 1V on 1.5H). MRK concluded that a volume of stone equal to 1-1/2 times the volume required to extend the slope protection to the expected depth of degradation provided an economic and efficient method of protecting the revetment against damage by undercutting and was sufficient to withstand parallel flow conditions (Reference 9).

37. A team consisting of personnel from MRD and MRK conducted the 6th Periodic General Inspection of Milford Dam in May 1977 (Reference 11). As part of this inspection the outlet channel riprap was examined. The team found the rock protection on the channel side slopes was performing satisfactorily (Figure 24) even though the streambed had degraded nearly 6 ft. Progressive armoring was developing over the entire length of the channel bed, and the upstream one half to two thirds of the channel bed was armored to the point where no additional degradation was expected. No areas of weakness in the toe protection were noted during this inspection (Figure 25).

38. An inspection of the four toe test sections revealed some

exposed areas between individual rocks. This was typical of the slope below the 12-in.-thick horizontal blanket in Test Section 3 (Figure 26) and isolated areas over Test Sections 1 and 2 (Figure 27). Good coverage of riprap on the slope below the 30-in.-thick horizontal blanket was noted in Test Section 4 (Figure 28). No areas of exposed sand could be found in this test section. A substantial growth of willows, cottonwoods, and other woody vegetation had developed over Test Section 3 (Figure 29). The team concluded that this growth could retard the downward movement of rock from this test section and that in order to retain a valid test, the vegetation should be removed. Breakdown of stone on the left bank near the waterline was noted. This was due to exposure to the sun during the winter and the resulting range of temperature variation that the riprap experienced; however, the extent of deterioration was not considered sufficient to affect the integrity of the revetment.

39. The WES Section 32 team visited the site on 18 September 1978. Figure 30 shows upstream and downstream views of the revetment in the outlet channel. The team found the side-slope revetment to be performing as designed; no apparent failures were noted. Later visits to the site by MRK through January 1980 indicate that the revetment continues to perform well.

Tributary (Little Timber Creek) to the Black Vermillion River at Frankfort, Kans. (Mile 0.0 to 0.3)

40. Little Timber Creek is a right-bank tributary of the Black Vermillion River at Frankfort, Kans. The Black Vermillion flows into the Big Blue River 11 miles downstream from Frankfort (28 miles upstream from Tuttle Creek Dam). Little Timber Creek drains a 13.5 square-mile watershed that is characterized by well-defined stream channels and gently rolling uplands. There are no long-term gaging or sediment records available for Little Timber Creek. The upstream average annual sediment yield is estimated to be 1,000 to 3,000 tons per square mile (Reference 4).

41. During the period 1903-1959, Frankfort experienced 18 damaging floods and numerous lesser overflows of streams surrounding the city. Prior to 1903, severe floods occurred notably in 1844 and 1859, but reliable information on these early floods is not available. During the 30 May 1959 flood of record the entire business district and about one third of the residential area of Frankfort (235 acres) were inundated to depths that at some locations exceeded 6 ft. Between 1903 and 1960, at least five other floods reached elevations within 18 in. of the 1959 flood.* The severity of these flash floods depended chiefly on the timing of crests of Little Timber Creek, the Black Vermillion River, and

* Data on the damaging floods were compiled from newspaper references to flood heights, areas inundated, relative depths of flooding, and from observed high-water marks.

West Fork (right-bank tributary of the Black Vermillion that enters the stream 2.1 miles downstream from the Little Timber-Black Vermillion confluence); thus, all floods did not reach comparable depths in all parts of the floodplain (Reference 12).

42. The 1958 Flood Control Act (Public Law 85-500) authorized improvements in the vicinity of Frankfort which featured a 3.4-mile-long levee along the east, south, and west sides of the low-lying portion of the city (Figure 31). This levee had incorporated into its design 3 ft of freeboard above the design flood flow of 43,000 cfs on the Black Vermillion. Other components of this project included channel improvements for 0.3 mile of Little Timber Creek and 1.5 miles of the Black Vermillion; a new bridge for the Missouri Pacific Railroad over Little Timber Creek; raising the Union Pacific Railroad bridge over the Black Vermillion River; and gated drainage outlets through the levee. Construction was initiated on 9 March 1962 and the project was transferred to the city of Frankfort for operation and maintenance on 24 October 1963. The total Federal cost was \$1,271,025 with the non-Federal cost estimated at \$122,000.

43. Improvements on Little Timber Creek included the construction of a new channel beginning at the Fourth Street Bridge (Figure 31) and extending downstream to the mouth. Prior to construction, the thalweg of the existing channel dropped from el 1129.5 ft at the Fourth Street Bridge to el 1123.0 ft at the Missouri Pacific Railroad bridge (Figure 31), with an average bed slope of 10.5 ft/mile. Construction of a new channel following the route shown in Figure 31 would have increased the slope to 21.1 ft/mile; thus with the design discharge of 2,200 cfs in the 14-ft bottom width channel and 1V-on-2H side slopes stream velocities approaching 10 fps would occur, corresponding to a Froude number of 0.73 (Reference 12).

44. Experience on other projects in MRK had shown that new channels cut in soils characteristically similar to those found in the Frankfort area were subject to excessive bed or bank scour. This erosion often resulted in the undermining and failure of bank slopes when a Froude number of approximately 0.5 was exceeded. To alleviate this problem on Little Timber Creek, MRK proposed to construct two grade-control structures (locally called ditch checks) at locations 547 ft and 1,322 ft above the mouth of the stream (Figure 31). These structures would reduce the bed gradient to an average slope of 8.45 ft/mile. With these structures in place, computations for the design discharge indicated that a maximum average velocity of approximately 7.0 fps would occur, with a corresponding Froude number of 0.43, which MRK considered compatible with the fat clays (CH) and medium to lean clays (CL) found throughout the Little Timber project reach.

45. Further MRK studies indicated that a savings of \$2,500 could be made over the use of rock ditch checks by using twin steel-sheet pilings at each check, with grouted stone between the pilings (Figures 32 and 33). This plan was adopted by MRK. The specifications for the No. 8 gage galvanized steel-sheet piling complied with the provisions of

the American Railway Engineering Association Specification 1-4-6. The sheet piling was placed with a pile driver. The stone in the 36-in.-thick horizontal layer across the channel bottom (Figure 33) was specified to have a maximum diameter of 24 in. and to be graded from coarse to fine with a maximum weight of 1,000 lb, and with 40 percent of the stone weighing more than 100 lb. The 18-in. maximum diameter riprap used on the side slopes was specified to weigh approximately 250 lb with no stone weighing more than 300 lb. The stone was placed with a grapple bucket. After placement the riprap surface was grouted. MRK reports that the cost for placement of the two grade-control structures is not separable from the total project cost. The two structures were collectively selected as a Section 32 existing site.

46. In October 1977, a team from MRK inspected the condition of the improvements on Little Timber Creek and found that the structures had been performing their intended function; however, a scour hole had developed downstream from ditch check 1 (Figure 34), and the grouted rock revetment had been undermined and was breaking off at ditch check 2 (Figure 35). The damage to the grouted rock has been repaired by the city of Frankfort. The WES Section 32 team inspected the Frankfort site on 19 September 1978 and found the grade-control structures to be performing as designed (Figure 36). MRK inspections through 16 April 1979 indicate that the ditch checks continue to prevent channel degradation.

Big Blue River near Marysville, Kans. (Mile 76.4)

47. Prior to 1973, flooding on the Big Blue River had caused considerable bank erosion upstream from the county bridge located one-half mile west of the town of Schroyer, Kans. (5 miles southwest of Marysville, Kans.). At that time, the bridge was used extensively as a farm-to-market road and U. S. mail route. An inspection of the bridge and its approaches by the County Engineer after flooding in October 1973 indicated that flanking of the structure was imminent. As a result the Marshall County Commission requested assistance from MRK under Section 14 of the Flood Control Act of 1946. In response, the following actions were considered by MRK: (a) no Federal action; (b) arrest the bank erosion at the present structure; and (c) construct another bridge in the immediate area. The analysis concluded that with the no Federal action alternative, bank erosion would eventually result in failure of the bridge. Failure of the structure would create a hardship in this rural area, since this is the only bridge crossing in an 11-mile reach. Action (b) was considered the more feasible of the construction alternatives since the cost of building a new bridge was estimated at \$215,000 (1975).

48. There are no discharge or suspended-sediment data for the Schroyer reach; however, data are available for the USGS gaging station at Barneston, Nebr. (mile 107.2). The daily discharges of record (1932

to the present) are maximum 57,000 cfs, mean 771 cfs, and minimum 1 cfs (Reference 13). MRK operated a suspended-sediment sample collection station at the same location from September 1959 through September 1972. Daily suspended-sediment loads of record were: maximum 357,800 tons (2 March 1966), mean 3,943 tons, and minimum 1.0 ton (16 August 1964). The maximum annual suspended-sediment load of record was 3,619,067 tons (water year 1965); the average annual suspended-sediment load was 1,439,107 tons. The average annual sediment yield in the area upstream from the Schroyer Bridge is 500 to 1,000 tons/square mile. Soils in this reach consist mostly of fine sands and silts in the bed and banks with some gravel in the bed and in lenses on the bank. The streambed gradient through this reach is 0.5 ft/mile.

49. Due to funding delays and right-of-way problems, construction of the Schroyer Bridge protection project was not undertaken until June 1977; the project was completed the following month. The final design consisted of 700 ft of fencing attached to railroad rail posts with three rock-dike tie-backs and 200 ft of rock revetment at the upstream end of the fencing (Figure 37). The purpose of this configuration was not only to redirect the flow, but also to induce deposition of a berm and thus reestablish the bank line along the fence.

50. The upstream revetment and the three dikes were constructed by placement of 1,500 tons of quarry-run stone. The construction specifications required that no more than 5 percent of the stone could be under 1/2-in. diameter, with the maximum stone weight being 500 lb. The upstream revetment was placed at the angle of repose of the stone. The zero crest width dikes were 6 ft in height with no specified bottom width; dike numbers 1-3 were 45, 150, and 125 ft in length, respectively (Figure 37).

51. The 10-ft-long fence posts were fabricated from salvage railroad rail and set 4 ft 8 in. in the ground on an 8-ft spacing. The posts were stabilized with three 1/2-in. galvanized-steel cables, which traversed the fence line at the bottom, middle, and top of the posts, respectively. The cables were passed through holes burned through the rails, and then permanently positioned with a cable clamp on either side of the hole. The fencing was then attached to the rails and cables with two strands of twisted No. 12 galvanized-steel wire (Figure 38). The fence fabric was 2- x 4-in. V-mesh No. 12 galvanized-steel wire. The base of the fence was buried to a depth of 1 ft, leaving 5 ft above the finished earth surface. The final cost (1977) of the project was \$39,895.21. The completed bank protection works were designated as a Section 32 existing site.

52. After completion, the project experienced three periods of high flow in 1977. Some minor damage occurred when a large tree was deposited on the fence during one of these discharges (maximum estimated flow during these three events was 16,000 cfs). High flows in July 1978 undermined a section of the fence causing it to fail (Figure 39). MRK indicates that a small windrow of rock at the base of the fence would probably have prevented this failure.

53. This project was visited by the WES inspection team on 19 September 1978. With the exception of the fence failure noted above, the project was performing as designed (Figure 40). Deposition was occurring behind the dikes and vegetation was becoming established. At the time of the WES visit, the bridge had been condemned by the Marshall County Engineer as a result of debris damage to the right abutment during early 1978. The bridge failed during high flows in early 1979; the steel span structure was washed downstream to the next point bar. Marshall County has no plans to replace the structure until Federal funding becomes available.

Deadman's Run and Antelope Creek at Lincoln, Nebr.

54. Deadman's Run and Antelope Creek are right-bank tributaries of Salt Creek (Salt Creek miles 26.0 and 28.0, respectively) at Lincoln, Nebr. (Reference 14). Salt Creek is a right-bank tributary of the Platte River entering that stream near Ashland, Nebr. Deadman's Run rises east of the Lincoln city limits and flows northwesterly 7 miles where it enters Salt Creek just downstream from the Chicago and Northwestern Railway bridge in north-central Lincoln (Figure 41). The 10.5-square-mile watershed is located entirely within Lancaster County, Nebr., having a length of about 6 miles and a maximum width of 2.5 miles. Topographically, the watershed is divided at the Holdredge Street Bridge. Downstream from this crossing, the streambed gradient is approximately 14 ft/mile; however, upstream the gradient ranges from 30 to 40 ft/mile. Watershed elevations in the Deadman's Creek project area range from 1,117 to 1,195 ft.

55. The headwaters of Antelope Creek are located near Cheney, Nebr. (Reference 14). From Cheney, Antelope Creek flows in a northwesterly direction 10 miles to join Salt Creek at the State Fair Grounds in Lincoln (Figure 41). The basin length is 8 miles with an average width of 2.5 miles. The total area of the drainage basin (entirely within Lancaster County, Nebr.) is 14 square miles, of which 5.4 square miles is controlled by Antelope Creek Dam, constructed by the Corps of Engineers in 1962. The basin's topography ranges from moderately rolling hills in the lower reaches to steeply rolling hills in the upper reaches. The streambed gradient averages 16 ft/mile from the confluence with Salt Creek to Antelope Creek Dam; above this impoundment the gradient increases to 28 ft/mile. Watershed elevations in the Antelope Creek project reach range from approximately 1,120 to 1,215 ft.

56. There is no long-term gaging information available for Deadman's Run. MRO records list the floods of 2 and 14 June 1951, and the flood of 25 June 1963, as being major events. During the flood of 2 June 1951, an area equivalent to approximately 40 city blocks was inundated with approximately 50 residences being underwater to some extent; no discharge or damage estimates were made. The 14 June 1951 storm flooded 43 city blocks between the northern edge of the University

of Nebraska and the Chicago, Rock Island and Burlington Northern Railroad tracks (Figure 41); the peak discharge was estimated to be 1,500 cfs. Floodwaters remained in the basin for about 5 hours and inundated 52 homes. Damages resulting from this flood event were estimated (1951) to be \$47,000. The 25 June 1963 storm in Deadman's Run was referred to as being "intense" (Reference 14); although no discharge estimates were made, near bank-full flow was observed. Damages were confined to the Cornhusker Highway Bridge (Figure 41) and were estimated (1963) to be \$27,000.

57. The USGS operated a stream-gaging station on Antelope Creek at Lincoln (mile 0.7) from 28 June 1958 through 30 September 1962. Daily discharges of record were: maximum 2,800 cfs (10 July 1958), mean 4.46 cfs, and minimum, no flow (7 and 21 December 1958) (Reference 13). The 1958 flood of record inundated 135 homes and a number of business establishments; however, no damage estimates are available (Reference 14). Prior to the period of record, there were a number of floods that occurred in the Antelope Creek Basin including those of 1908, 1910, 1940, 1950, 1951, 1952, and 1957. The two most significant of these were 14 June 1951 and 27 June 1952. During the 1951 event, heavy rains over the basin caused inundation of about 760 acres. The flood conditions were aggravated by bridge constrictions and floating debris. Damages to 92 commercial buildings, 298 homes, streets, and railroads were estimated to be \$472,000 (1951). The storm of 1952 was less severe, damaging only five commercial structures and ten residences.

58. No suspended-sediment load data are available for either stream; however, average annual sediment yields over the basins range from 1,000 to 3,000 tons/square mile (Reference 5). Surface soils along the streams are generally fat clays (CH) and lean and sandy clays (CL).

59. Both Deadman's Run and Antelope Creek are naturally meandering streams; however, these two creeks are not in their original channels due to residential and commercial encroachments. During the development of adjacent areas, the channels were straightened into new alignments which resulted in higher channel velocities. The increased velocities tended to develop wider streambeds and to encourage channel degradation. This erosion created 20-ft-high banks at many locations that were nearly vertical, which in turn caused much concern among local residents not only because of the loss of property, but because of the safety hazard. To mitigate these problems, the Lower Platte South Natural Resources District (LPSNRD) developed a program to control the erosion and water flow. Initially, a concrete slab rectangular channel liner was placed in a small test reach; however, weep hole plugging caused excessive uplift pressure resulting in damage to the structure. Because the concrete liner proved to be unstable, gabions were selected torevet the banks of both streams not only to provide adequate bank drainage, but to allow placement of nearly vertical structures that would minimize top-bank property losses.

60. Initial bank stabilization to sustain a 100-year flood event began in 1969 and continues to the present. Through 1979, 11 construction reaches have been completed in Deadman's Run, having a total

project length of 15,201 ft (Figure 41 and Table 1). Six reaches have been completed in Antelope Creek and one is now under construction; these seven reaches will total 11,537 linear ft in length. The projects in both streams were designed for LPSNRD by Clark Enerson Partners of Lincoln, with the onsite construction being completed by various local contractors. The 18 construction reaches have been collectively selected as a Section 32 existing site.

61. Prefabricated gabion cages have been marketed in Europe for many years; however, gabions for the construction of bank protection works in the United States have been used widely only in the past 20 years. The basic element of the gabion revetment is the cage or "basket." The cage is a wire mesh structure divided by diaphragms into cells (Figure 42). The gabions used for this project were obtained from Maccaferri Gabions, Inc. The mesh was fabricated from U. S. Steel Wire Gauge #11 zinc-coated, galvanized wire. The tensile strength of the wire was specified to be in the range of 60,000 to 85,000 psi, with a minimum zinc coating on the wire of 0.80 oz/sq ft. The maximum dimension of the mesh opening could not exceed 4-1/2 in. and the area of the mesh opening not larger than 8 sq in. The wire mesh was required to have sufficient elasticity to permit elongation of the mesh equivalent to a minimum of 10 percent of the length of the section under test without reducing the gauge or tensile strength of the individual wires to a value less than that for similar wire, one gauge smaller in diameter. The gabions were shipped flat and wired together onsite (Figure 42). After assembly, the cages were filled with stone. The stone was specified to have not more than 5 percent by weight of the total material passing a 3-in. sieve and not more than 10 percent by weight of the material retained by a 12-in. sieve. The maximum weight for any one stone could not exceed 40 lb and have no dimension less than 3 in. or greater than 16 in. In addition, the stone was specified to be of a composition suitable to withstand abrasion, to be nonfriable, and to be resistant to weathering and freeze-thaw actions.

62. Prior to gabion placement, the bank was shaped and the streambed compacted. A gabion support apron was then placed on the compacted material (Figure 43). The aprons served to protect the toe of the revetment and to distribute some of the load of the gabion baskets which would be stacked vertically on the apron. The aprons have dimensions of 1 ft vertical, 3 ft wide (parallel to bank), and 6, 9, or 12 ft long, depending on how much of channel bed was to be covered (at locations where scour was possible, such as culvert outlets, bridge piers, etc, the entire bottom of the channel was lined).

63. After completion of the support apron, the first course of gabions was placed. The baskets were filled with stone by machine, and then hand-arranged to minimize voids. The stone on the front face of the gabion (Figure 43) was stacked to present a pleasing appearance (Figure 44). Noncorrosive wire was used to fasten the gabions together and to the support apron to further strengthen the structure. A minimum of a 2-ft width of fill material was placed behind the gabions and compacted. To minimize overall project costs, only one course of gabions

was used where possible. The top face of the gabion course was then covered with fill material which was then sloped back at 1V on 3H toward intersection with the natural terrain profile. For reaches where the surface area along the top bank was at a premium (residences, commercial concerns, etc.) gabion courses were stair-stepped as high as practical to maximize the available top-bank surface area (Table 1 and Figure 45). No filter (sand, gravel, fabric) was generally used below the apron or between the gabions and the bank. The exceptions were known moist banks where leaching of the soil through the gabions was possible and at intersections of the stream with drainage culverts where fabric was placed under the support apron in the area of the culvert discharge impact (Celanese Mirafi 140 or Dupont Typar were recommended for use by Clark Enerson Partners, although local contractors were at liberty to make their own selection).

64. The upper bank was seeded with reed canary grass (5.0 lb/acre) in all areas except parks, where a bluegrass mixture (50 lb/acre) was used. Although reed canary grass has a superior root system, the bluegrass has a much more pleasing appearance and is more easily maintained. After seeding, the exposed area was mulched with clean native hay at the rate of 2-1/2 tons/acre.

65. At the time of the WES inspection visit, the projects were performing well. Maintenance has been limited to vegetation clearance (Figure 46) and replacement of stone removed by vandals; maintenance costs are not available. The project has not been tested by the 100-year design flood (bank-full condition), but was subjected to a 20-year flood in September 1977 with no apparent damages. No discharge estimates are available for this event; however, bank-full conditions were not approached at any location. The gabions were overtopped through several reaches.

Floyd River at Sioux City, Iowa (Mile 0.1 to 1.6)

66. The Floyd River, located in northwestern Iowa, is a left-bank tributary of the Missouri River at Sioux City, Iowa (Missouri River mile 731.2). The Floyd River Basin is comparatively long and narrow with a total drainage area of 956 square miles (Reference 15). For much of its course, the river flows in a broad alluvial valley, with bottomlands being 3,000 to 3,500 ft wide in the vicinity of Sioux City. Although the valley walls are gently sloping farther upstream, the valley is flanked by high, steep-sided loessial bluffs as the stream approaches its confluence with the Missouri.

67. No discharge or sediment data have been collected on the Floyd River at Sioux City; however, the USGS has operated a gaging station on the Floyd at James, Iowa (mile 10.7), since 1934. Daily flows of record are: maximum 71,500 cfs (8 June 1953), mean 173 cfs, and minimum 0.9 cfs (10-22 January 1977) (Reference 16). MRO operated a daily suspended-sediment sample collection station at James from 16

March 1954 through 30 April 1957 (Reference 4). Operation of this station was resumed by the USGS from 1 October 1968 through 30 September 1973 (Reference 16). Daily suspended-sediment loads of record were: maximum 171,000 tons (25 May 1954), mean 1,230 tons, and minimum, no load (a number of days during water years 1956 and 1957). The maximum annual suspended-sediment load of record was 799,804 tons (water year 1971); however, the total suspended-sediment load for water year 1954 could possibly have exceeded the 1971 value based on the 6-1/2-month total of 729,587 tons. The average annual load was 449,561 tons. Average annual sediment yields range from 6,000 to 10,000 tons/square mile in the vicinity of Sioux City; however, yields decrease rapidly to 250-300 tons/square mile above James (Reference 4).

68. Improvements on the Floyd River at Sioux City were authorized by the Flood Control Act of 1958 (Public Law 85-500). This legislation provided for construction of a new 2-mile-long channel from the relocated Missouri-Floyd confluence to the 18th Street Bridge (Figure 47); straightening and enlarging 4 miles of existing channel upstream from the 18th Street Bridge to the upper end of the project (north of 46th Street); construction of 9 miles of levee along both banks of the river; and construction of four new railroad bridges across the channel (Reference 17). The Public Law 85-500 appropriations were supplemented by a 1962 congressional authorization for twin bridges at the Interstate Highway 29 crossing, 300 ft upstream from the Missouri-Floyd confluence. Construction of the improvements on the Floyd at Sioux City began in June 1961 and was completed in July 1966. Total cost (1966) was \$18,356,000, including \$6,800,000 of non-Federal funds.

69. The Floyd River flood control project design included a high-velocity rock-lined channel to carry a maximum discharge of 71,500 cfs (equal to the flood of record) below the level of the adjacent natural ground. The rock-lined channel would lie in the relatively erosion-resistant clays of the Missouri River floodplain; however, a few feet below the design bed grade were deep deposits of highly erodible sands and gravels. With the erodible material lying so close beneath the channel bottom, there was a high potential for deep localized scour and extensive bed degradation during high-velocity flows. In the event that extensive degradation occurred, it could cause serious damage by undermining bridge piers and abutments and the toes of revetted side slopes. Another consequence of degradation would be excessive drawdown of the water surface and destructive velocities in the proposed leveed earth channel upstream from the rock-lined reach. MRO therefore considered it necessary to place a series of grade stabilization structures that would deter head cutting in the event degradation occurred and could create sufficient head losses to maintain the design water-surface elevations (Reference 18). A brief model study at the University of California at Berkeley indicated that a row of sheet piling across the channel bed with some form of rock protection might adequately retard the development of head cutting and create sufficient losses to maintain the desired upstream water surface. Further model studies were conducted at the University of Iowa to develop criteria for adequate rock protection for the sheet piling (Reference 18).

70. The design of the lower mile of the project was dictated, to a large extent, by severe limitations on the available right-of-way and by the numerous streets and railroads crossing the project. The large number of crossings and the adjacent railroad yards required that the water-surface elevation be kept as low as possible in order to avoid the prohibitive cost of raising the elevation of the bridges and railroad yards. In addition, the bed grade of the realigned channel had to meet the bed grade of the existing natural channel at the head of the reach, and at the same time the grade could not be so steep through the project reach that excessive velocities for discharges up to 71,500 cfs would be produced. Normally these stringent requirements would dictate the use of a high-velocity concrete-lined channel; however, the cost of such a channel was prohibitive. The selected design called for a trapezoidal channel (with an erodible bed) and rock protected 1V-on-2.5H side slopes. The bottom width of the proposed channel would narrow from 280 ft at the Missouri River to 100 ft at the upper end of the project. Using this design, MRO estimated that the average stream velocity through the reach would be slightly in excess of 14 fps for the design discharge of 71,500 cfs. Velocities would be greater than 10 fps for discharges more than 23,000 cfs, which is approximately the 25-year flood (Reference 18).

71. After the model tests were completed, computations based on the results indicated that five sills spaced approximately 2,000 ft apart in the lower 1-1/2 miles of the project reach would effectively maintain desired water levels in the channel despite considerable bed scour; further even if the bed was to degrade 10 ft, the water surface would be maintained within 1 to 2 ft of the design elevation with no degradation (Reference 18). As a result of these tests, MRO placed five sheet piling and rock sills at miles 0.1, 0.4, 0.8, 1.1, and 1.6 upstream from the new Missouri-Floyd confluence. These sills, completed in 1965, have been collectively designated as a Section 32 existing site. These structures provide a 4-ft/mile bed gradient through the improved channel (elevations through the project reach vary from approximately 1,070 ft at the Missouri-Floyd confluence to el 1,080 at mile 1.6).

72. The sharp-crested sills were constructed using single rows of sheet piling at the design bed elevation (Reference 19) (Figures 48 and 49). The steel-sheet piling conformed to ASTM A-328-54, "Standard Specifications for Steel-Sheet Piling." The pile sections were required to be of the continuously interlocking type throughout their entire lengths when in place. The properties of the sections were specified as follows:

Type of Section	Nominal Web Thickness in.	Weight per Square Foot of Wall* lb
DA-27	3/8	27
SA-23	3/8	23

* Weight per square foot may not vary over 2.5 percent above or below the value shown.

73. The stone used to complete the structures was required to meet the following specifications (Figure 49):

	Stone Size, lb		
	Maximum	Median	Minimum
Derrick stone	6,000	4,000*	2,000
36-in.-thick riprap	600	280	20
18-in.-thick riprap	300	180	20
15- or 12-in.-thick riprap	150	80	20

* Half of the derrick stone by weight should be larger than this size.

Neither the breadth or thickness of any piece of stone was allowed to be less than one third its length. The bedding material met the following gradation

Sieve Size	Percent Passing by Weight
3 in.	60-100
3/4 in.	0-60
No. 4	0-30

74. Since the sills were constructed, the new channel has experienced yearly maximum flows of 700 to 17,000 cfs. The peak flow was recorded in April 1969, which was well under the design flow of 71,500 cfs. At the time of the WES inspection visit on 20 September 1978, the sills were performing well (Figures 50 and 51). MRO inspections through September 1979 indicate that all structures are sound and continue to function as designed.

West Fork Ditch at Onawa, Iowa (Mile 0.0 to 9.2)

75. The Little Sioux River Basin Flood Control Project was authorized by the Flood Control Act of 1947 as modified in 1954 (Preference 20). The project was designed to protect 188,000 acres of highly productive farmland and several small communities located along the Little

Sioux River from its mouth to the community of Smithland, Iowa. Construction began in 1956 and was finished in 1966. The completed project provided for placement of 138 miles of levee and 62 miles of channel improvements which included the enlargement and straightening of existing channels or the excavation of new channels. Since 1959, when the project became partially operational, it has prevented flood damages estimated at more than \$19,580,000. The total cost of the project was \$18,483,000 (1972), including \$3,000,000 derived from non-Federal sources. Local interests operate and maintain the project.

76. West Fork Ditch was dug by the Monona-Harrison Drainage District in the early 1900's. This ditch now drains the watershed previously emptied by the west fork of the Little Sioux. Flow from the natural channel of the West Fork is diverted into the ditch near Holly Springs, Iowa. The West Fork Ditch discharges at the intersection of Wolf Creek, the Garretson Outlet Ditch, the Monona-Harrison Ditch, and the Little Sioux/Monona-Harrison Diversion Channel (Figure 52). When the western portion of the Little Sioux Basin is in flood, the discharges of West Fork Ditch, Garretson Outlet Ditch, and Wolf Creek are passed into the Monona-Harrison Ditch and the Diversion Channel (and thus to the Little Sioux). When the Little Sioux is in flood, part of its flow is passed into the Monona-Harrison Ditch via the diversion channel. Because the runoff characteristics of the basin differ at opposite ends of the diversion channel, rarely do flood crests occur simultaneously. Generally, the crest in the western part of the basin occurs much earlier than in the eastern portion.

77. The USGS operated a gaging station on the west fork of the Little Sioux at Holly Springs, Iowa (mile 11.4), from April 1939 through September 1969. The station was relocated on the Iowa State Highway 141 Bridge at Hornick, Iowa (mile 9.2) in July 1974 with data collection continuing to the present (Figure 52). The daily discharges of record are maximum 12,400 cfs, mean 94.3 cfs, and minimum 0.2 cfs (Reference 16). The maximum observed flow velocity has been 8 fps. The discharges measured at the Hornick gaging station are considered typical of the West Fork project reach, although some additional discharge is accepted from Farmers Ditch at mile 6.9. MRO operated a suspended-sediment sample collection station at Holly Springs from August 1957 through September 1967. Daily suspended-sediment loads of record were: maximum 204,000 tons (14 June 1967), mean 1,178 tons, and minimum zero (on several days). The maximum annual load of record was 911,200 tons (water year 1962); the average annual load was 429,854 tons (Reference 4). Average annual sediment yields in this region are 6,000 to 10,000 tons/square mile, which are among the highest yields in the entire Mississippi River Basin. The surface soils through the project reach are a mixture of fat clays (CH), clayey silts (ML), and lean clays (CL).

78. The West Fork levee system was completed in June 1964. By the late 1960's channel degradation had become a serious threat to the levees and several internal drainage structures. Five low rock sills were placed in 1971-1972 at approximately equal intervals along the

channel (at headcut locations) between mile 0.0 and the Iowa State Highway 146 Bridge at Hornick (Figure 52). The sills were constructed from rock placed on filter fabric. The fabric was specified to be monofilament yarn woven in a rectangular pattern; however, the type of fabric used is not known. The sills were constructed as a weir set at the design channel elevation, with a downstream apron 3 ft below the weir crest; riprap was placed along the channel side slopes for an additional 40 ft downstream from the apron (Figures 53 and 54). The stone used to construct the low-rock sills was specified to be 24 in. in diameter (maximum dimension) with the following gradation:

<u>Weight, lb</u>	<u>Percent Lighter by Weight</u>
1,500-2,500	100
1,000-1,800	50
500-1,000	15

The completed project provided for a bed-gradient fall of 1.5 ft/mile for a design flow of 11,100 cfs. The total cost (1972) for placement of the five sills was \$54,150.

79. High flows in early 1973 produced severe lateral erosion downstream from the sills which in turn threatened to fail the levees; in addition a county bridge below sill 2 was seriously threatened. Emergency work was required in April 1973. Setback levees were constructed at sills 3 and 4, and sill 2 was removed (\$52,000). In October 1973, the remaining sills were modified according to the results of limited model studies conducted at Mead Laboratory. The modification included two rows of sheet piling driven to refusal in the weir section to provide positive (uniform) cross-channel flow (Figure 55). In addition, the modification called for placement of riprap and shaping the downstream channel side slopes according to the recommendations of the model studies (Figures 56 and 57). The sheet piling complied with the guidelines provided by ASTM A 328-70, "Steel-Sheet Piling." The stone riprap used for the repairs had the same specifications as that stone used for construction of the original sills (previous paragraph). The total cost of these modifications was \$91,000 (1973). The four modified sills were designated as existing Section 32 sites.

80. Topographic surveys conducted since 1973 indicate that the channel is in equilibrium. No headcutting has been observed and the four modified sill structures are in an as-built condition. At the date of the WES inspection visit, the project was performing well (Figure 58).

Little Sioux River at Onawa, Iowa (Mile 5.7)

81. The Little Sioux River has headwaters in Jackson County, Minn., and flows in a southwesterly direction until the stream discharges into the Missouri River main stem near mile 669.2. The

Monona-Harrison Drainage District dug an equilizer ditch between the Little Sioux and the Monona-Harrison Ditch in the early 1900's (Figure 59). As a result, the Monona-Harrison Ditch captured the discharge of the Little Sioux such that little or no flow was passed below the diversion. As part of the Little Sioux River Basin Flood Control Project (paragraph 75), the equilizer ditch was closed and the Little Sioux was rebuilt downstream from the diversion. During flood periods, discharge can now be exchanged between the Little Sioux and Monona-Harrison Ditch via an upstream diversion channel (paragraph 76 and Figure 52).

82. The USGS has operated a gaging station on the Little Sioux near Turin, Iowa (mile 13.5), since January 1958 (Figure 59). The daily discharges of record (1958 to the present) are: maximum 30,000 cfs (19 February 1971), mean 1,085 cfs, and minimum 22 cfs (10-22 February 1959) (Reference 16). MRO operated a daily suspended-sediment sample collection station at the same location from March 1959 through July 1969. Daily suspended-sediment loads of record were: maximum 1,120,000 tons (1 June 1959), mean 10,414 tons, and minimum 12 tons (on several days) (Reference 4). The maximum annual suspended-sediment load of record was 6,694,200 tons (water year 1962); the average annual load was 3,801,130 tons. Average annual sediment yields over the lower Little Sioux Basin are 6,000 to 10,000 tons/square mile (Reference 5).

83. Prior to the placement of three grade-control structures at the mouth of the Little Sioux in 1959 (Figure 59),* channel degradation had progressed approximately 3.5 miles upstream from the mouth. Between 1959 and 1964 the headcutting advanced another 2.5 miles or a total of 6 miles. The degradation had thus progressed to the point where MRO no longer considered it practical to control the advance by increasing the stage in the lower reach of the stream through the use of additional grade-control structures at the mouth. The proposed method of halting the headcutting was to place a control structure just downstream of the upper limits of the serious erosion. The grade-control structure (designated #4 in Figure 59) consisted of a 50-ft-wide rectangular drop structure in the central channel flanked by upstream rock sills on 158-ft-wide berms extending to the levees (Figure 60). The structure was built using conventional riprap following recommendations from a WES model study (Reference 21). The structure (completed in 1964) was designed to withstand a discharge of 35,000 cfs at stream velocities up to 18 fps; with the structure in place the bed gradient through this reach was adjusted to 1 ft/mile.

84. The structure was designed such that when flows greater than 8,000 cfs occurred, the upstream channel berms were overtopped, causing overbank flows downstream of the drop. This flow reentered the downstream channel over the stilling basin riprap. Additional downstream degradation caused more lowering of the tailwater than was anticipated, which resulted in severe problems with the reentrant overbank flows. Damage to the riprap due to high flows in April 1965 (27,100-cfs maximum

* Part of the construction under the Little Sioux River Basin Flood Control Project (paragraph 75).

daily flow) required placement of grouted derrick stone (800-1,000 lb) to repair the structure (Figure 61). Further flows undermined sections of the grouted stone revetment. In 1969, a 150- by 50-ft gabion mattress was placed on the left and right banks of the stilling basin downstream of drop structure (Figure 60). Each mattress consisted of interwoven rock-filled baskets with 12- x 3- x 1-ft dimensions. The outer edge of each mattress consisted of a single row of 6- x 3- x 3-ft baskets to tie down the toe of the mattress. The gabions were specified to have physical properties equivalent to those gabions sold by Terra Aqua, Inc., of Reno, Nev.; no other specifications are available except that the gabion wire was galvanized. The stone placed in the baskets was required to have a gradation of 4 to 8 in. The total cost for placement of the two mattresses was \$25,000 (1969). The mattresses were damaged in 1973 by return flows and some of the baskets had to be replaced; in addition, a 2- to 3-in. layer of grout was placed over the mattress surfaces to prevent debris from hanging up in the mesh and subsequently breaking the wire. Also the grout prevented rust; although the wire was galvanized, fishermen often built fires on the exposed mattress, which removed the protective coating, thus making the metal susceptible to oxidation. The total cost for the 1973 repairs was \$22,000. Again, in 1975, several baskets had to be replaced and grouted (\$6,000). The two gabion mattresses were selected as a Section 32 existing site.

85. At the time of the WES inspection visit (20 September 1978), failures were observed on the right-bank grouted stone section (Figure 62) and on the left-bank mattress. The mattress damage included loss of grout (Figure 63) and failure of part of the mattress under its own weight, due to scour under the mattress (Figure 64). Snowmelt runoff in March 1979 resulted in overbank return flows that partially failed the right-bank mattress (Figure 65) and completely failed the left-bank mattress (Figure 66). Flows during this period were mostly between 10,000 to 20,000 cfs with a peak flow of 24,000 cfs. MRO does not plan to repair either mattress; the entire structure may have to be redesigned. Although the structure has successfully stopped further upstream headcutting, the weir width was apparently not sufficient. MRO is currently considering placement of two additional weirs on either side of the existing structure, at slightly higher elevations.

102 River (East Fork) at Bedford, Iowa

86. The East Fork of the 102 River rises in the hills of northeastern Taylor County, Iowa, flows past the county seat at Bedford, and then joins the West and Middle Forks near Hopkins, Mo., to form the 102 River. The East Fork was reworked by local landowners both upstream and downstream of Bedford in 1946 and 1947. Approximately 2 miles of channel was improved northeast of Bedford beginning at a point 1,000 ft above the Iowa State Highway No. 2 Bridge (Figure 67) and extending upstream; 6-1/2 miles of channel was improved downstream of Bedford beginning 2 miles south of the city and extending to Hopkins, Mo. A short,

300-ft-long segment of improvement was also made in the city of Bedford.

87. Discharge data have been taken at the USGS gaging station near Bedford since September 1959 (2.4 miles downstream from the State Street Bridge (Figure 67)). Daily discharges of record through the present are: maximum 9,980 cfs, mean 49.9 cfs, and minimum, no flow (Reference 16). No reported suspended-sediment samples have been taken on the East Fork of the 102 River. Average annual sediment yields in the vicinity of Bedford are 3,000 to 6,000 tons/acre. Lean and sandy clays (CL) with some fat clay (CH) are found in both the bed and banks of the 102 River at Bedford.

88. MRK assistance in controlling flood flow on the East Fork was initiated in 1966 under authority of Section 205 of the 1943 Flood Control Act, as amended. The first contract was awarded on 16 August 1966; the project was completed on 16 October 1967. The improvement consisted of straightening and widening the channel through Bedford and downstream to a point 2 miles south of the city. The project provided for 1V-on-3H side slopes, a 45-ft bottom width, and a bed gradient modification from 0.84 to 7.13 ft/mile which collectively increased the design capacity of the improved channel to 7,500 cfs. All trees and brush along the channel top bank and 50 ft landward of top bank were removed. Riprap was placed on both banks upstream and downstream of the low head dam (Figure 67); in addition, stone revetment was placed on the right bank from sta 62+00 to 67+50. The completed project provided protection for 100 urban acres and 300 acres of agricultural lands. The project was completed and transferred to the city of Bedford for operation and maintenance on 29 November 1967.

89. High flows (over 5,000 cfs) occurred in April 1969. At sta 62+00 through 67+50 a band of riprap and bedding about 20 ft wide was swept off the bank through the middle third of the slope. General degradation was found to be occurring along the entire length of the channel. A plan to rebuild the eroded slope, replace the riprap and bedding material, and stabilize the channel bottom with a drop structure at sta 61+60 was developed. Authority to proceed with the construction was received on 23 October 1972. On 29 December 1972, high flows damaged sheeting that had been driven at the drop structure construction site; in addition the channel degraded 6 ft through this reach. As a result, the structure had to be redesigned, and work was not completed until 12 September 1973.

90. Flooding in October 1973 accelerated erosion adjacent to three existing facilities. Although channel degradation and bank erosion had occurred prior to the flood, the increased degradation and erosion worsened to such an extent at these locations that their safety was classified as critical. MRK specified the following emergency measures (Figure 67):

- a. Slope repair and stabilization of the right bank adjacent to the Bedford Water Treatment Plant.
- b. Slope repair and stabilization of the left bank at the State Street Bridge.

c. Slope repair and stabilization of the left bank at the Bedford Sewage Treatment Plant.

91. MRK chose to stabilize the slope at these three problem areas using Fabriform, manufactured by Construction Techniques, Inc., Cleveland, Ohio (Figures 68-71). Fabriform is a double-walled woven nylon material which is filled with fluid fine-aggregate concrete. The fabric was specified to be of the filter point type (8 in. between points), thus providing capability for drainage. Prior to placement of the Fabriform mattress the banks were shaped with quarry-run stone (no gradation specified; Figure 72). After placement of the mattress on the prepared bank, grout was pumped into the Fabriform. The grout was required to have the following specifications per cubic yard: Portland cement 900 to 1,000 lb, aggregate 2,200 to 2,000 lb, and water 570 to 610 lb. Air entrainment was specified to be in the range 3 to 6 percent of the total mixture.

92. The contract for the revetment placement was let on 25 February 1974 and work was concluded on 22 May of the same year. The completed protection works (Figures 73-75) required 31,000 sq ft of Fabriform and 5,300 tons of quarry-run stone. The blankets were keyed into top bank with an earth-fill anchor trench (Figure 68). These three revetments, whose total cost (1974) was \$124,000, were collectively designated as a Section 32 existing site.

93. A storm event in excess of 8 in. occurred on 9-10 June 1974; a daily discharge of 4,930 cfs was recorded at the gaging station during this event. A 27 June 1974 MRK inspection indicated that the downstream 25-ft portion of the fabric-covered area at the water treatment plant had received considerable damage resulting from erosion of slabby rock from under the toe supporting the grout-filled fabric on the channel bottom. This erosion extended to a depth of approximately 5 ft and 10 to 20 ft riverward of the intersection of the slope with the channel bottom. The major damage was at the downstream end of the revetment where a fabric seam was torn 5 ft upslope. The State Street and sewage treatment plant revetments were intact and no damage was noted. A further inspection by MRK on 4 November 1976 indicated that further erosion under the revetment at the water treatment plant was in progress.

94. The WES inspection team visited this site on 21 September 1978. Although high flows had undercut the Fabriform at several locations resulting in parts breaking off at the toe (Figure 76) or subsidence of sections (Figure 77), most of the revetments were generally in good condition (Figure 78). During the March 1979 flood (2,000-cfs maximum daily flow), the drop structure (Figures 67 and 71) was undercut and failed (Figure 79). In an attempt to protect the upstream reach, MRK constructed a temporary dam at the site of the structure; the construction proceeded from the right bank across to the left bank. This caused a concentration of flow on the left bank (against the Fabriform mattress) during most of the afternoon of 25 March before closure was made. As a result, nearly all of the fines were washed out of the rock fill in this vicinity, creating a "pipe" under the Fabriform that was

parallel to the streamflow. Consequently, the flow continued to be concentrated against the left bank until the morning of 26 March. This resulted in a relatively large erosion hole on the left bank under the Fabriform. As a result, the undercut Fabriform had to be broken out on 26 March and rock fill placed in the void.

95. An inspection of the mattress at the water treatment plant during the same time period indicated that a large cavity had formed under the Fabriform up to top bank (Figure 80); this was a potentially serious situation because the corner of the plant was only 10 to 15 ft from top bank. An inspection conducted by MRK on 31 July 1979 indicated that a considerable portion of the Fabriform adjacent to the plant had been displaced. The city of Bedford has placed several gabions on the exposed bank which probably prevented a total undercutting of the toe. MRK repaired the revetment in February 1980 with grouted rock and gabions. Under the same contract a temporary grouted rock and gabion structure was placed adjacent to the sewage treatment plant, upstream from the original drop structure. The repaired revetment at the water treatment plant successfully withstood high flows in June 1980; however, the drop structure adjacent to the sewage treatment plant was lost. As a result, some of the Fabriform was undermined and collapsed into the cavity.

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Table 1

Summary of Gabion Placement in Deadman's Run and Antelope Creek

Construction Reach	Construction Schedule		Completed Cost* (\$)	Project Length, ft	No. of Gabion Courses**
	Started	Completed			
Deadman's Run					
1 - 48th to 52nd Streets	1970	1970	166,831	1,750	5+
2 - 52nd to 56th Streets	1970	1970	196,760	1,750	5+
3 - 56th to 60th Streets	1973	1974	218,747	1,950	1
4 - 60th Street to Cotner Blvd	1969	1970	85,346	1,750	1
5 - Gateway	1971	1972	86,245	1,800	1
6 - 66th to 70th Streets	1971	1972	139,463	1,800	2+
7 - 70th to "O" Streets	1971	1972	83,106	950	2
8 - 33rd & Huntington Streets	1973	1975	189,732	1,831	1+
9 - East Campus Bridge	1973	1974	89,850	800	2
10 - East Campus Bridge to 40th Street	1978	1979	63,311	500	1+
11 - Cornhusker Highway	1971	1972	22,452	205	2
				15,201	
Antelope Creek					
1 - 52nd to 56th Streets	1971	1971	30,741	930	1
2 - 52nd to Van Dorn Streets	1976	1977	53,795	535	1
3 - 48th to 52nd Streets	1977	1978	179,447	1,967	1
4 - South to 33rd Streets	1971	1973	268,849	3,600	2+
5 - 33rd to "A" Streets	1969	1972	148,623	1,955	2
6 - Nebraska State Fairgrounds	1973	1974	223,933	1,750	2
7 - Eden Park	1979	Inc.	79,750	800	-
				11,537	

* Includes cost for clearing, grubbing, excavation, backfill, grading, gabions and rock-fill material, seeding, and some sewer construction.

** A plus means that more gabion courses than the number indicated were used at some locations.

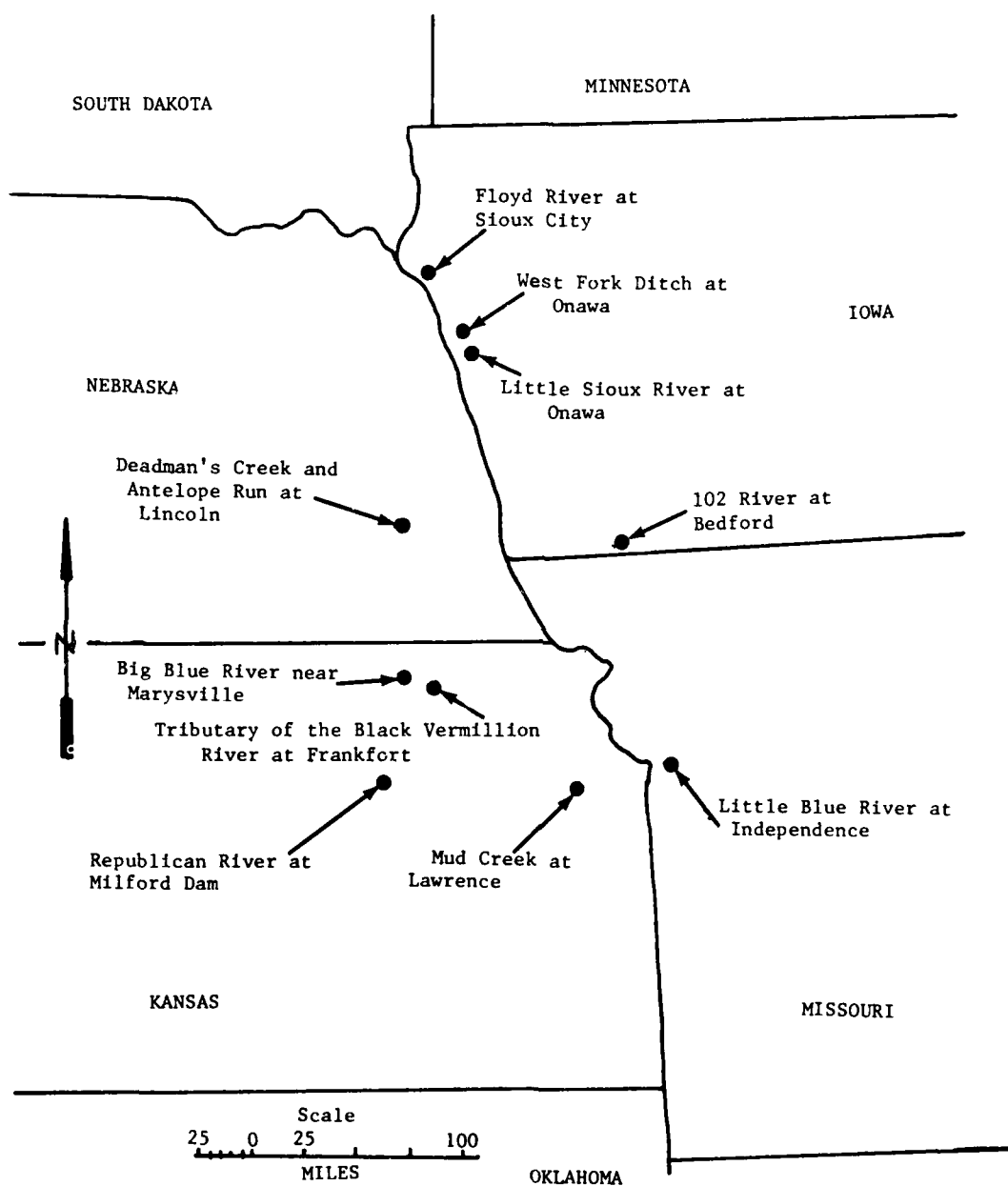


Figure 1. Section 32 Program existing sites in the Missouri River Division inspected by the WES evaluation team, 18-21 Sep 1978



The photograph shows a landscape with a body of water in the middle ground and a line of trees in the background. The foreground is dark and silty, suggesting a riverbank or a field of mud. The water is calm, and the trees are dense and dark. The overall scene is somewhat desolate and appears to be a natural, undeveloped area.

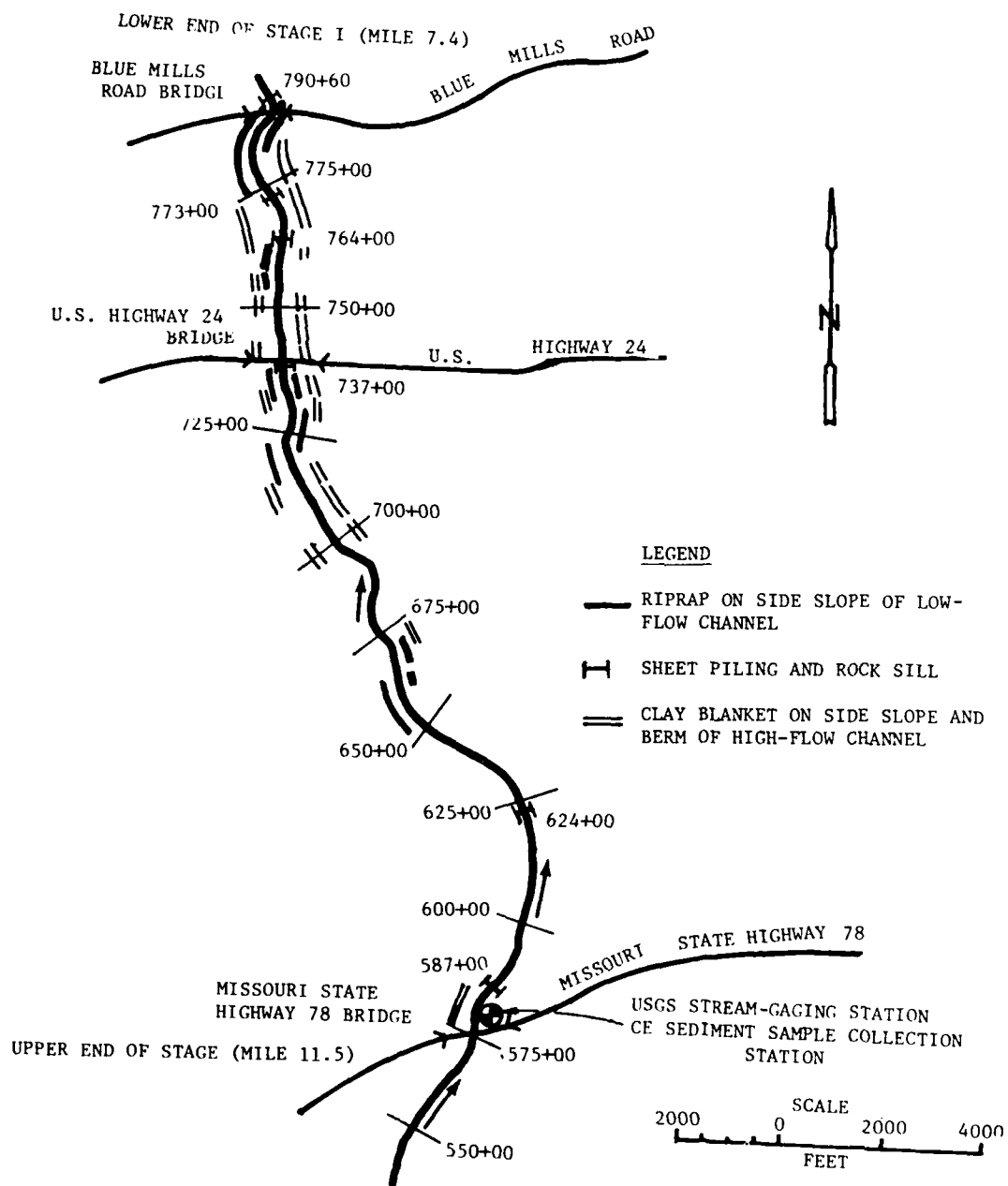


Figure 3. Little Blue River channel improvement at Independence, Mo., Stage I (sta 577+00 to 788+96)

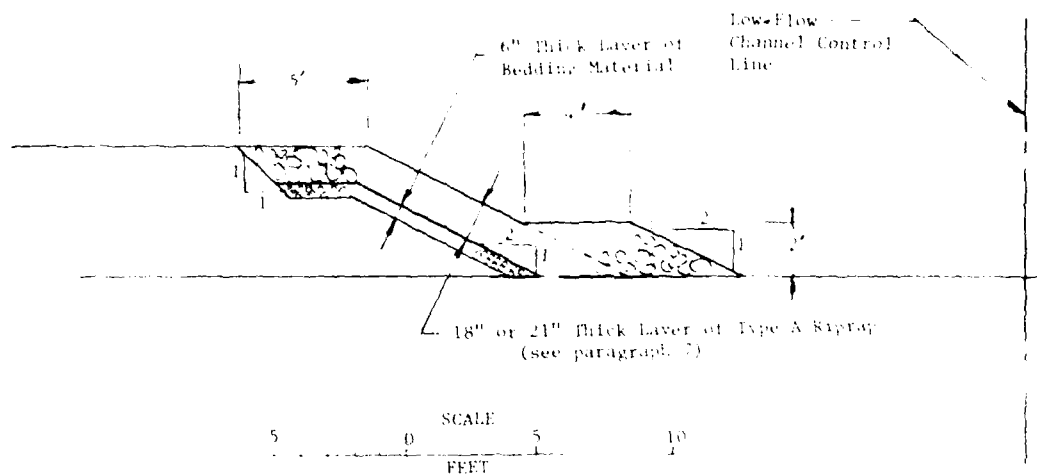


Figure 4. Little Blue River at Independence, Mo. Slope of low-flow channel protected by stone riprap (13 September 1945)



Figure 5. Little Blue River at Independence, Mo. Slope of low-flow channel protected by stone riprap (13 September 1945)

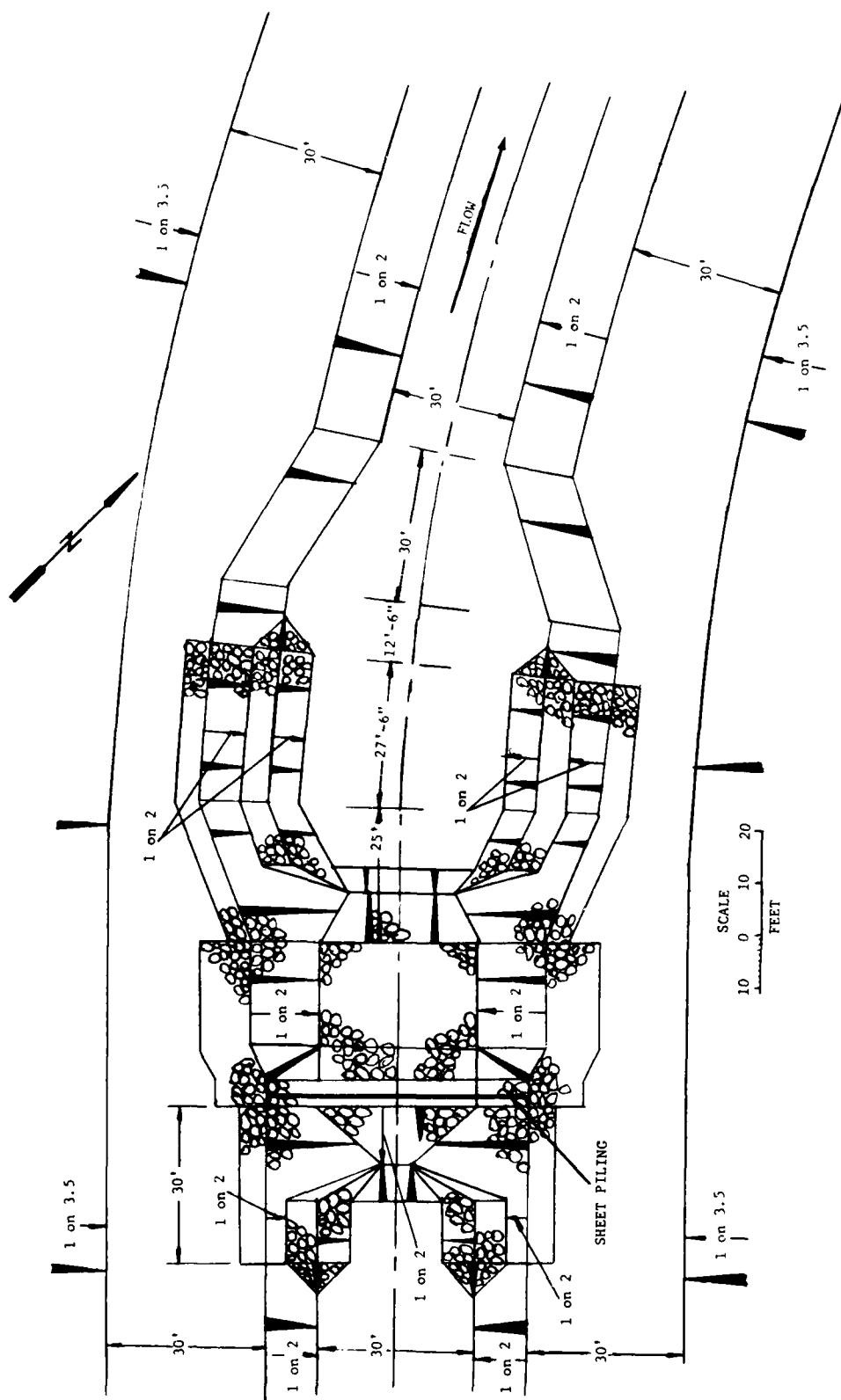


Figure 6. Little Blue River at Independence, Mo. Typical plan view of sheet piling and rock sills (adapted from Little Blue River Channel Improvement, Stage I, Plate 17, September 1974, MRK)

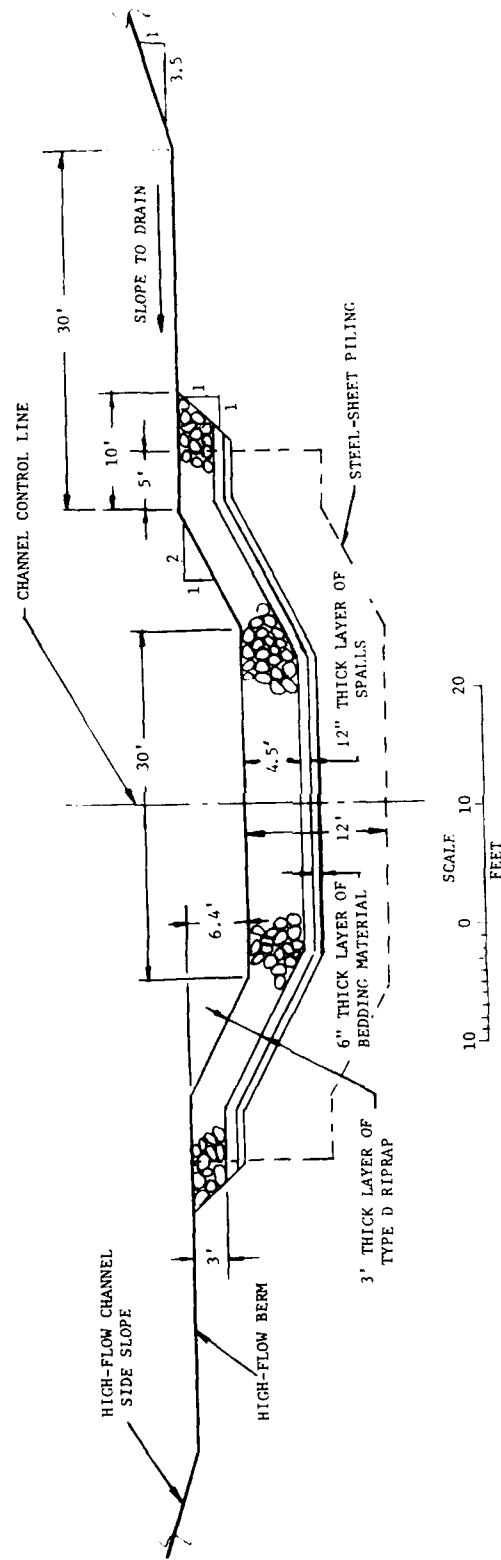


Figure 7. Little Blue River at Independence, Mo. Cross-sectional view of sheet piling and rock sills (adapted from Little Blue River Channel Improvement, Stage I, Plate 17, September 1974, MRK)



Figure 8. Little Blue River at Independence, Mo.
Interlocking sheet piling used for construction
of sills (18 September 1978)

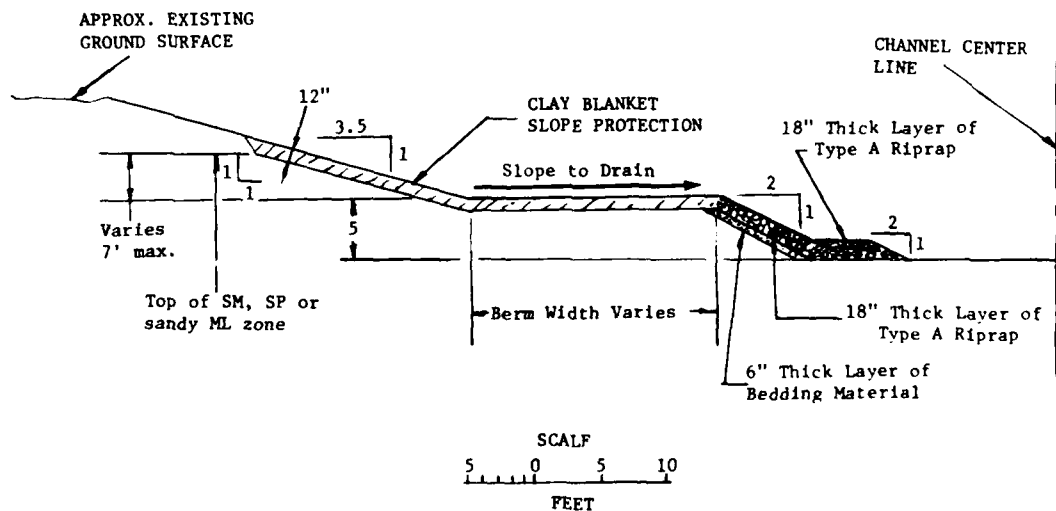


Figure 9. Little Blue River at Independence, Mo. Typical section of
clay blanket side-slope and berm protection (adapted from Little Blue
River Channel Improvement, Stage I, Plate 17, September 1974, MRK)



Figure 10. Little Blue River at Independence, Mo. Debris loading behind sheet piling and rock sill at sta 587+00 (18 September 1978)



Figure 11. Little Blue River at Independence, Mo. As a result of uncontrolled overbank drainage, clay blanket erosion has occurred at some locations (May 1977)



Figure 12. Little Blue River at Independence, Mo. At the time of the WFS inspection visit (17 September 1978) the overall project was in an as-built condition. Note that the grass has become well established on the high-flow channel side slope and berm; the low-flow channel riprap is also well covered. Grass mowing has sometimes been a problem due to frequent flooding and the wet condition of the berm.

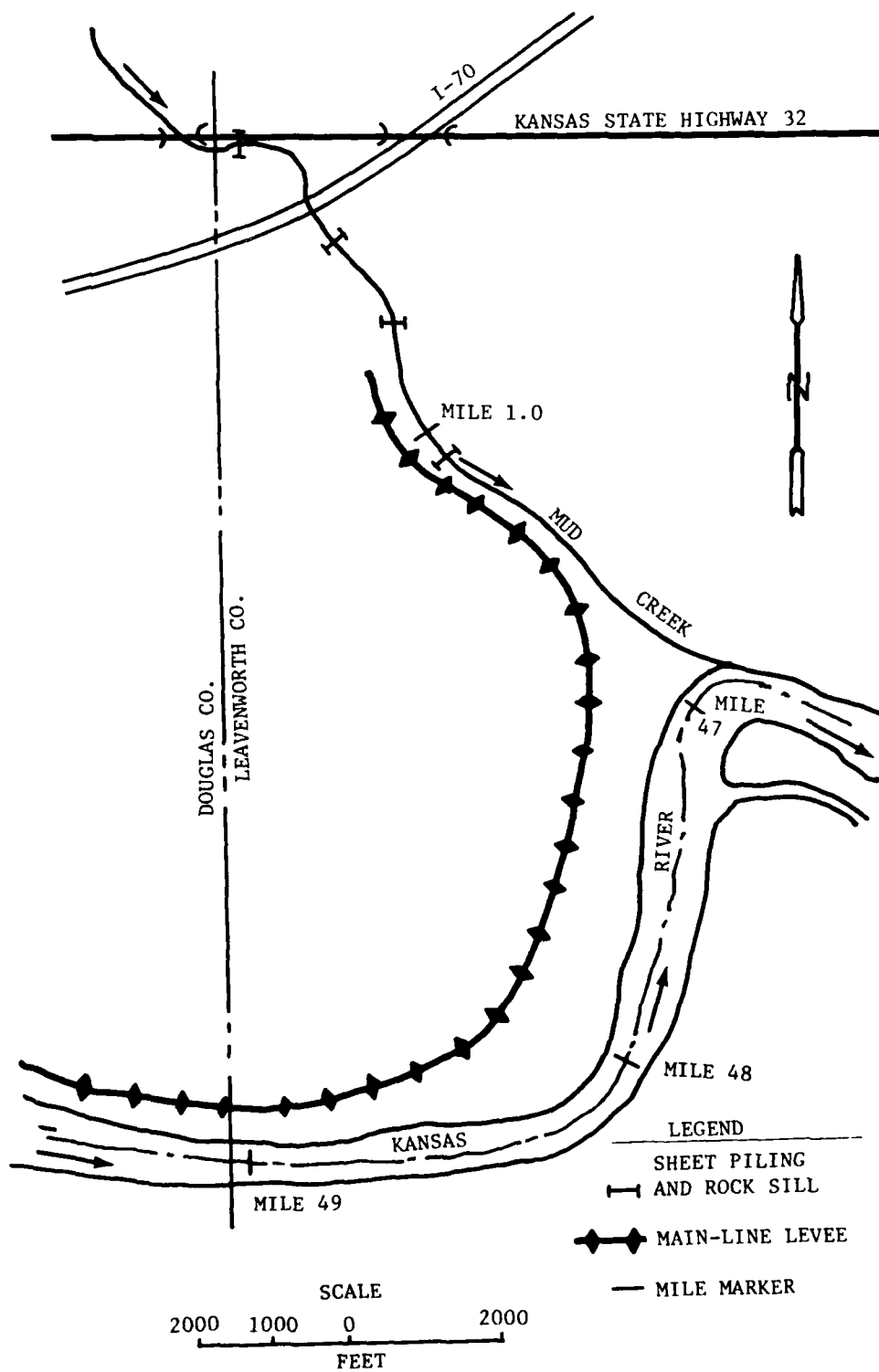


Figure 13. Mud Creek at Lawrence, Kans.

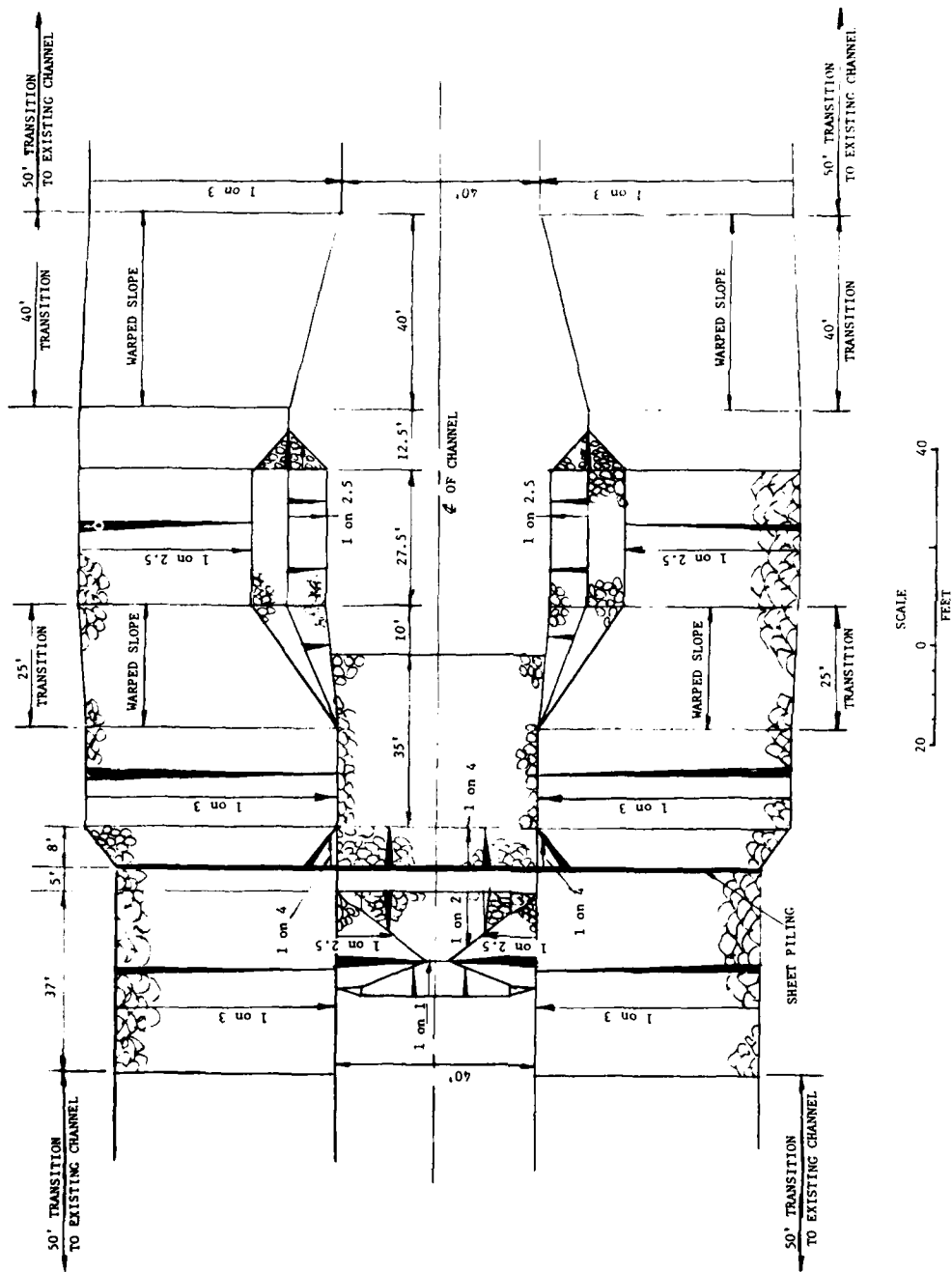


Figure 14. Mud Creek at Lawrence, Kans. Plan view at sheet-piling and rock sill structure (adapted from Lawrence, Kans., Flood-Control Improvements, Mud Creek Unit, Drawing 39, March 1976, MRK)

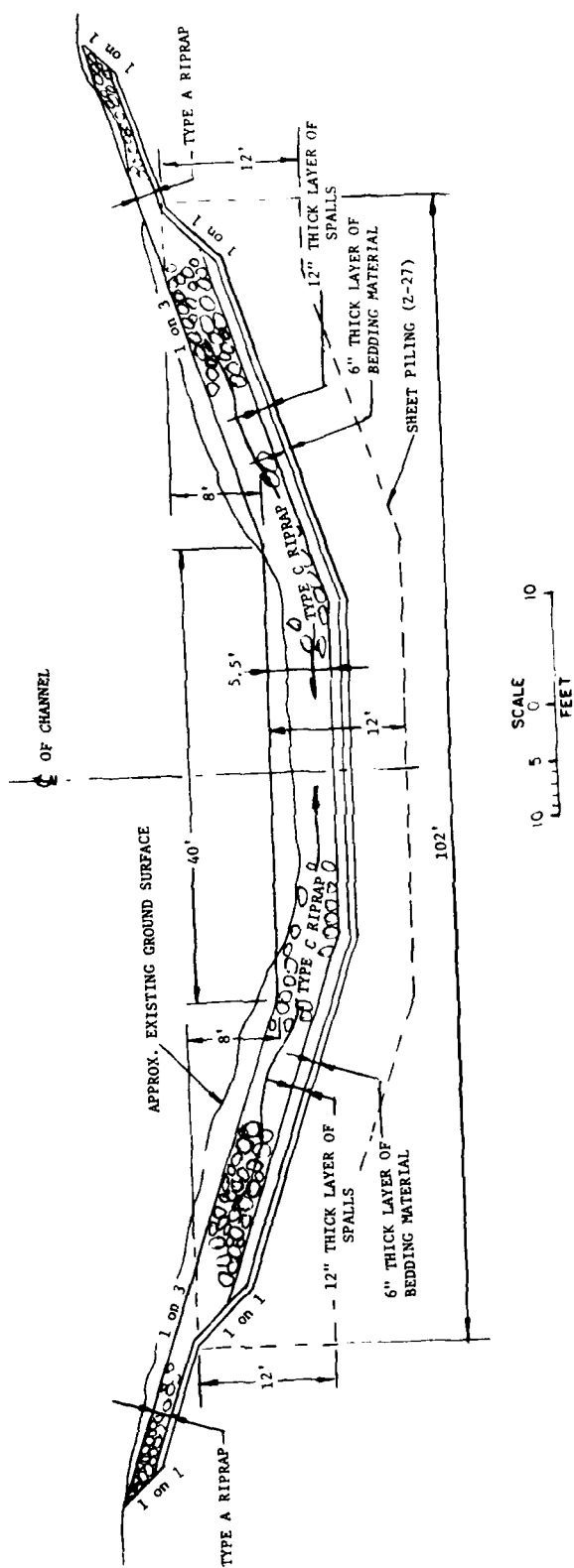


Figure 15. Mud Creek at Lawrence, Kans. Cross-sectional view of sheet-piling and rock sill structure (adapted from Lawrence, Kans., Flood-Control Improvements, Mud Creek Unit, Drawing 39, March 1976, MRK)



Figure 16. Mud Creek at Lawrence, Kans. Interlocking sheet piles (18 September 1978)



Figure 17. Mud Creek at Lawrence, Kans. Sheet piling and rock sill at mile 1.55 (18 September 1978)

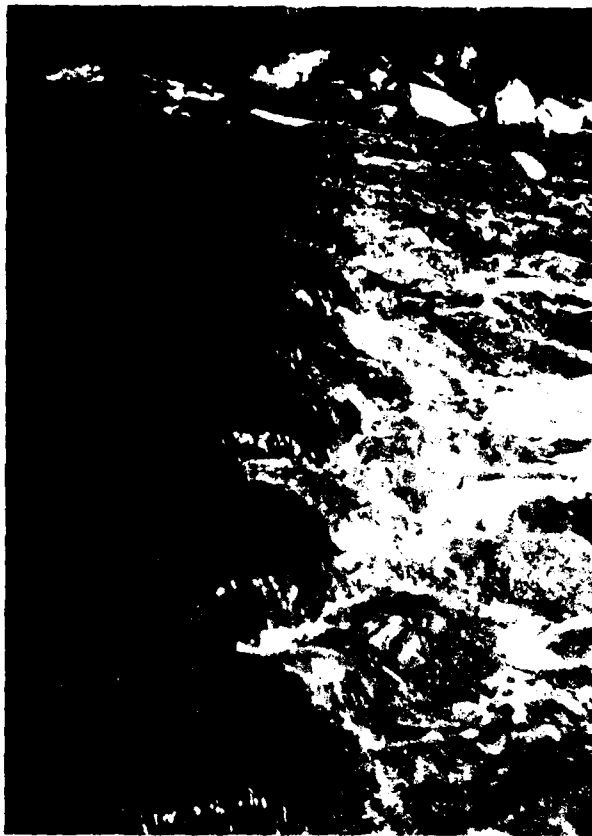


Figure 1. Map of the area, showing
the location of the site, and
the location of the site.

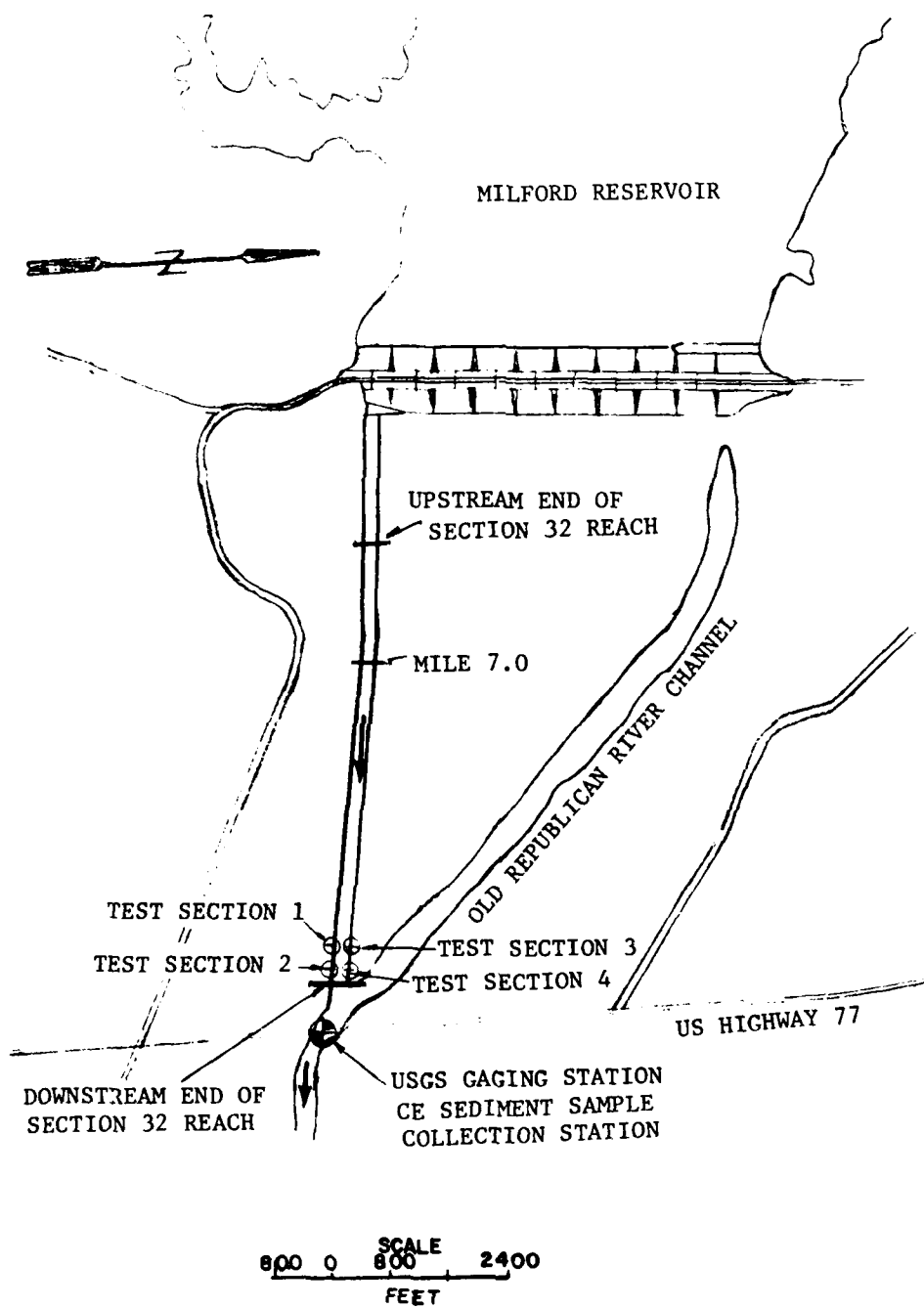


Figure 19. Republican River at Milford Dam Outlet Channel, Kans.

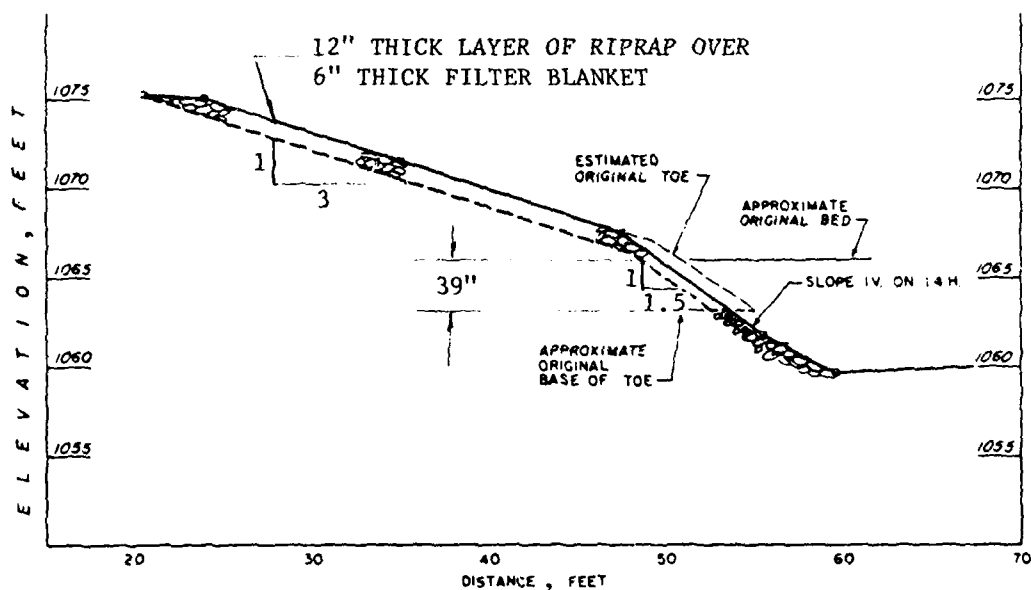


Figure 20. Republican River at Milford Dam Outlet Channel, Kans. Cross-sectional view of Test Section 1 shown as constructed in 1969 and as surveyed in 1974. (Adapted from Reference 9)

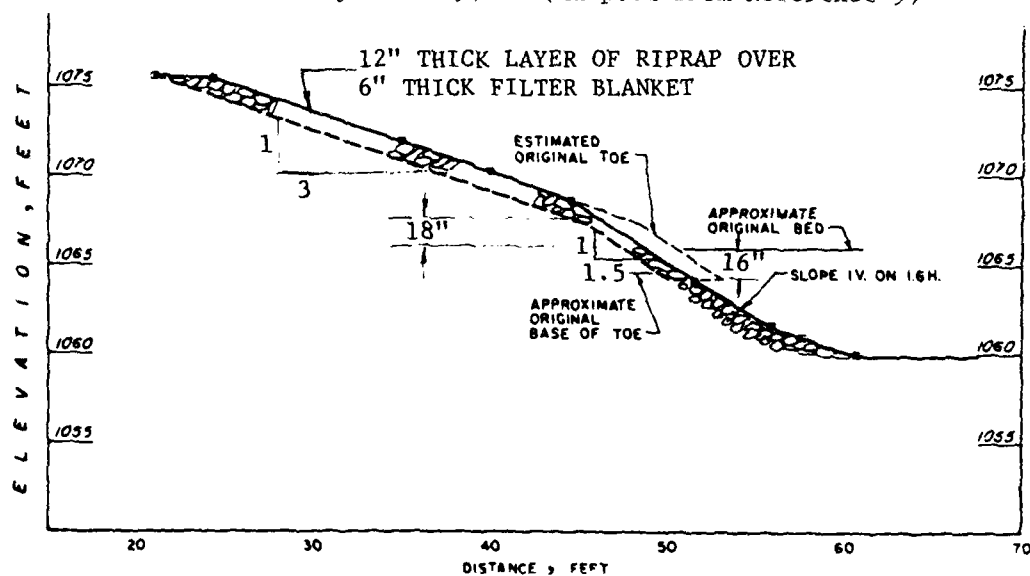


Figure 21. Republican River at Milford Dam Outlet Channel, Kans. Cross-sectional view of Test Section 2 shown as constructed in 1969 and as surveyed in 1974. (Adapted from Reference 9)

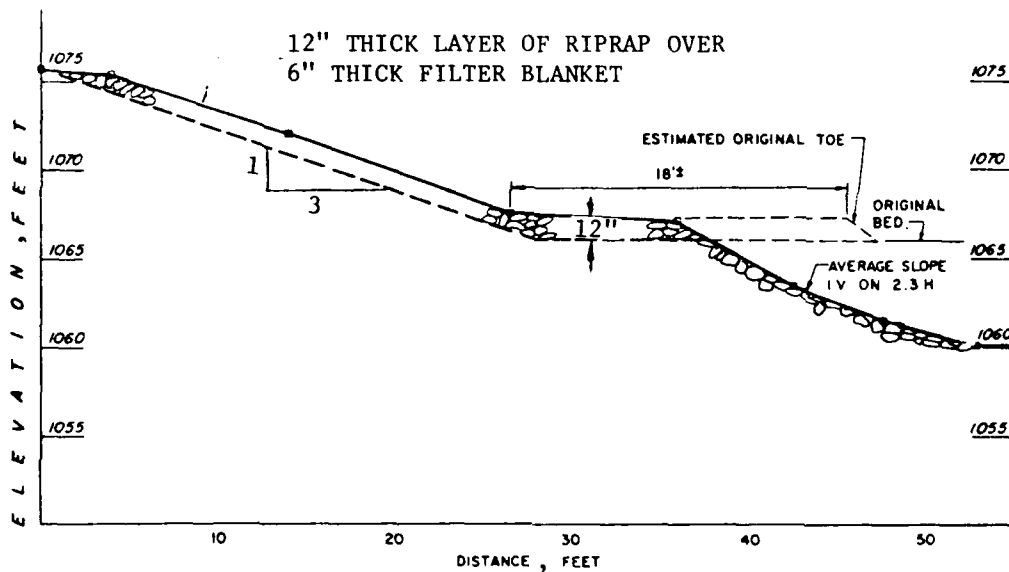


Figure 22. Republican River at Milford Dam Outlet Channel, Kans. Cross-sectional view of Test Section 3 shown as constructed in 1969 and as surveyed in 1974. (Adapted from Reference 9)

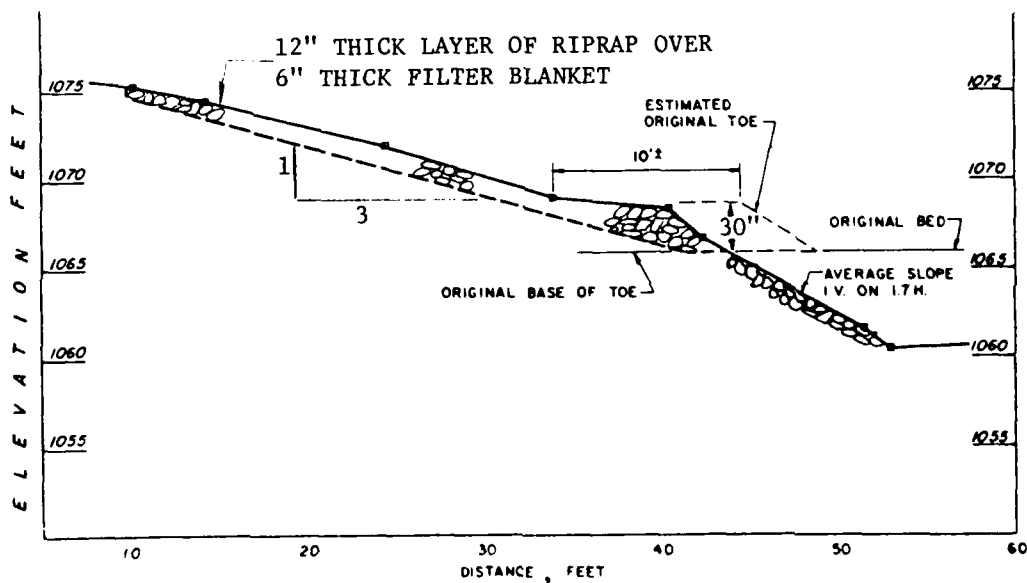


Figure 23. Republican River at Milford Dam Outlet Channel, Kans. Cross-sectional view of Test Section 4 shown as constructed in 1969 and as surveyed in 1974. (Adapted from Reference 9)



Figure 24. Republican River at Milford Dam Outlet Channel, Kans. General appearance of the middle reach of the outlet channel at the time of the May 1977 inspection. (Source: Reference 11)



Figure 25. Republic River at Milford Dam Outlet Channel, Kans. Typical view of the revetment toes as seen by the MRD-MRK inspection team in May 1977. The engineer in the photograph is standing on the horizontal blanket at the break in slope between the blanket and the slope to the water. (Source: Reference 11)

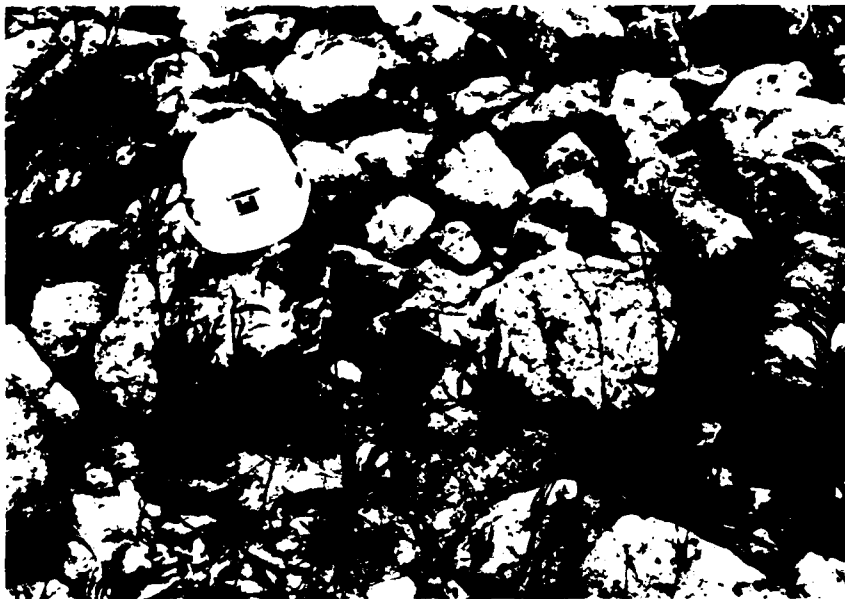


Figure 26. Republican River at Milford Dam Outlet Channel, Kans. Areas of exposed sand visible between individual rocks below the 12-in.-thick horizontal blanket in Test Section 3 (May 1977). (Source: Reference 11)



Figure 27. Republican River at Milford Dam Outlet Channel, Kans. The area behind the hard hat is typical of the isolated areas of exposed bedding material in Test Sections 1 and 2 (May 1977). (Source: Reference 11)



Figure 28. Republican River at Milford Dam Outlet Channel, Kans. There was good coverage of rock on the slope below the 30-in.-thick horizontal blanket at the time of the May 1977 inspection (Test Section 4). (Source: Reference 11)



Figure 29. Republican River at Milford Dam Outlet Channel, Kans. There was a substantial growth of woody vegetation over Test Section 3 at the time of the May 1977 inspection. The view is downstream toward the U. S. Highway 77 Bridge. (Source: Reference 11)



a. Upstream



b. Downstream

Figure 30. Republican River at Milford Dam Outlet Channel, Kans. Upstream and downstream views of the outlet channel revetment at the time of the WEC inspection visit. Note that the vegetation has been removed in the lower view; compare with Figure 29.
(18 September 1978)

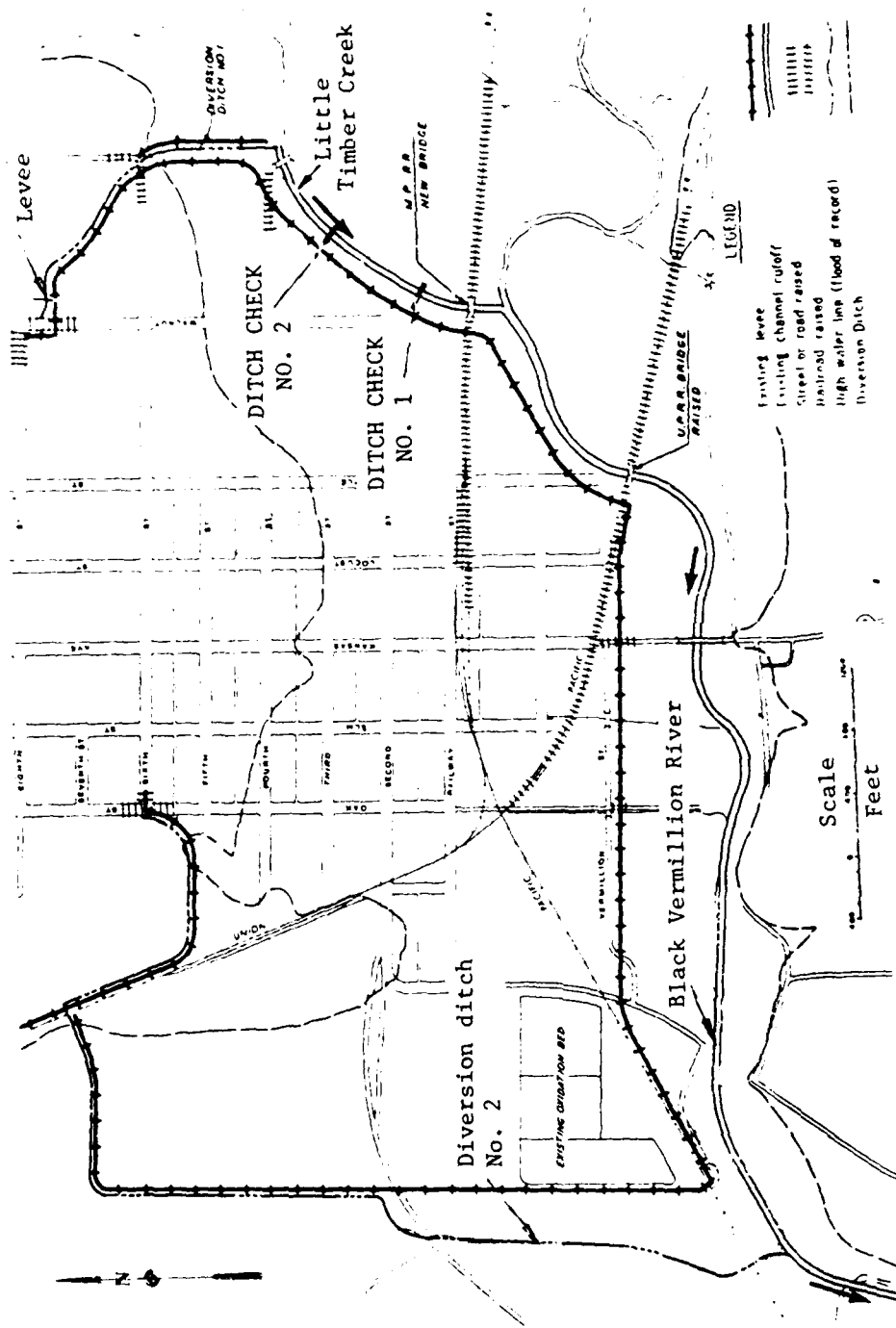


Figure 31. Little Timber Creek at Frankfort, Kans. (Source: Reference 12)

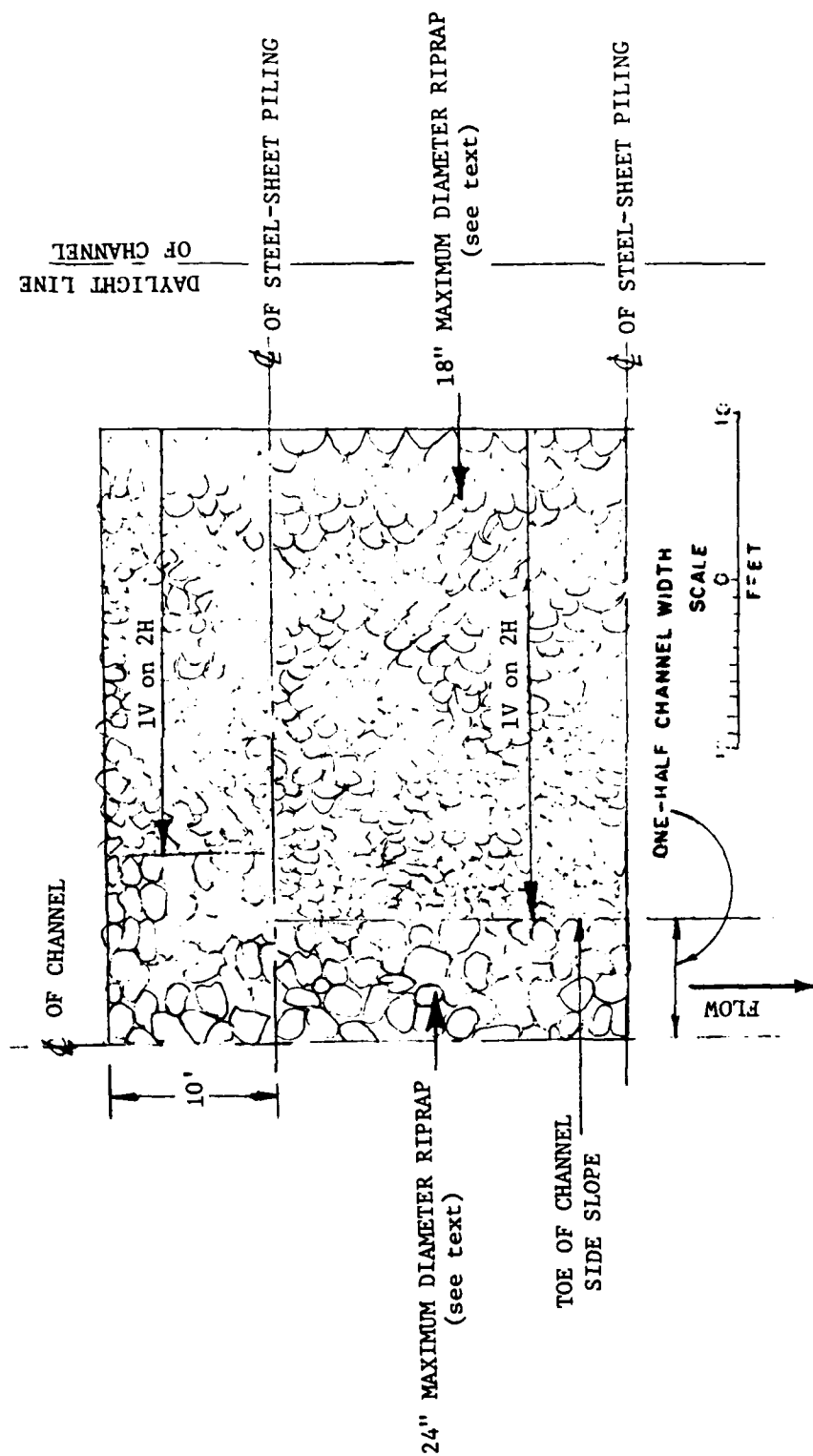


Figure 32. Little Timber Creek at Frankfort, Kans. Half plan of ditch checks (Source: Kansas River Basin, Frankfort, Kans., Black Vermillion River, Channel Profiles, Underground Explorations and Ditch Check Details, Sheet No. 55, MRK, October 1963)

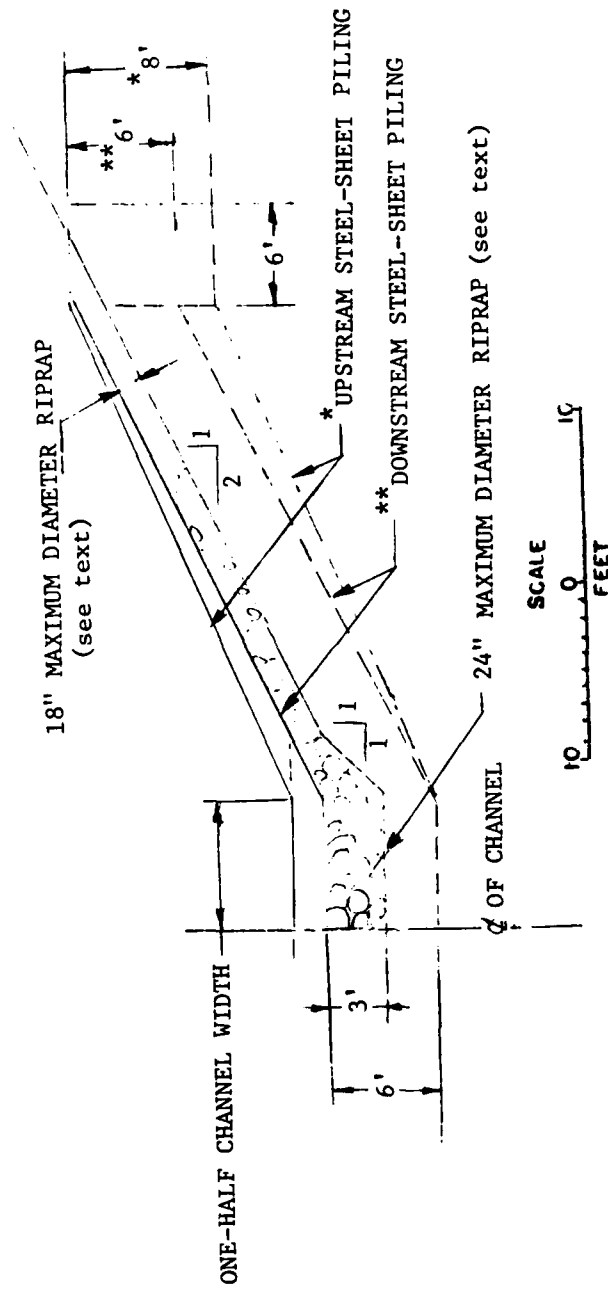


Figure 33. Little Timber Creek at Frankfort, Kans. Half section of ditch checks looking upstream (Source: Kansas River Basin, Frankfort, Kans., Black Vermillion River, Channel Profiles, Underground Explorations and Ditch Check Details, Sheet No. 55, MRK, October 1963)



Figure 24. Little Timber Creek at Frankfort, Kans. View of scenic hole
looking from dam ditch check 1 (October 1977)

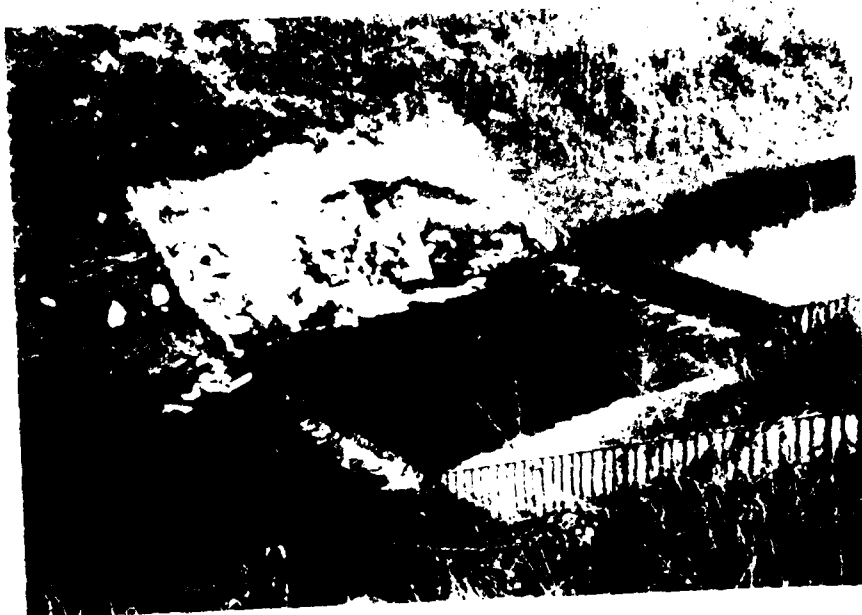


Figure 3. Little River Look at Bradford, Pa.
downstream view in winter of eroded rock from
ditch check. (Station 207)

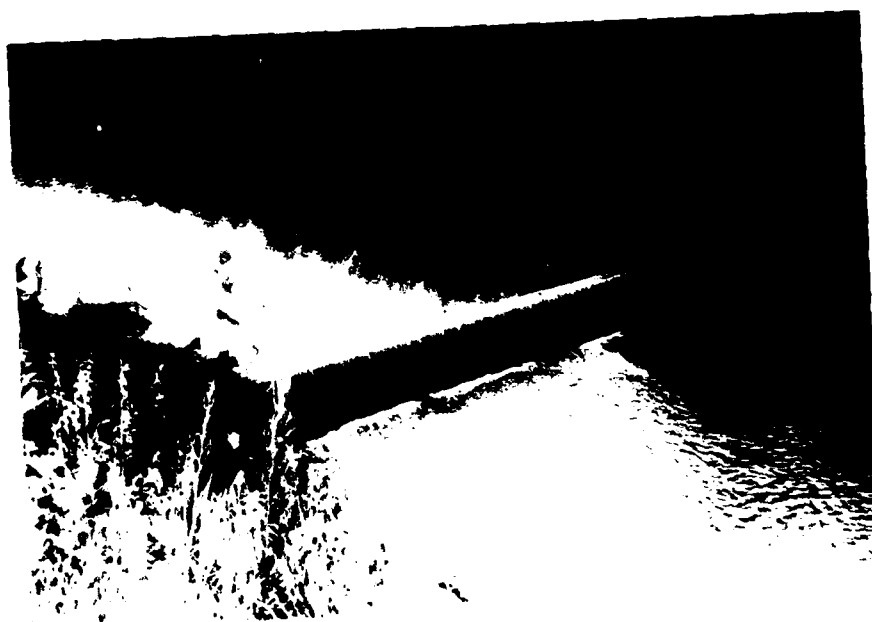


Figure 4. Little River Look at Bradford, Pa. The ditch
shows were perpendicular to a line of the flow of the WSC In-
spiration visit in September 1971.

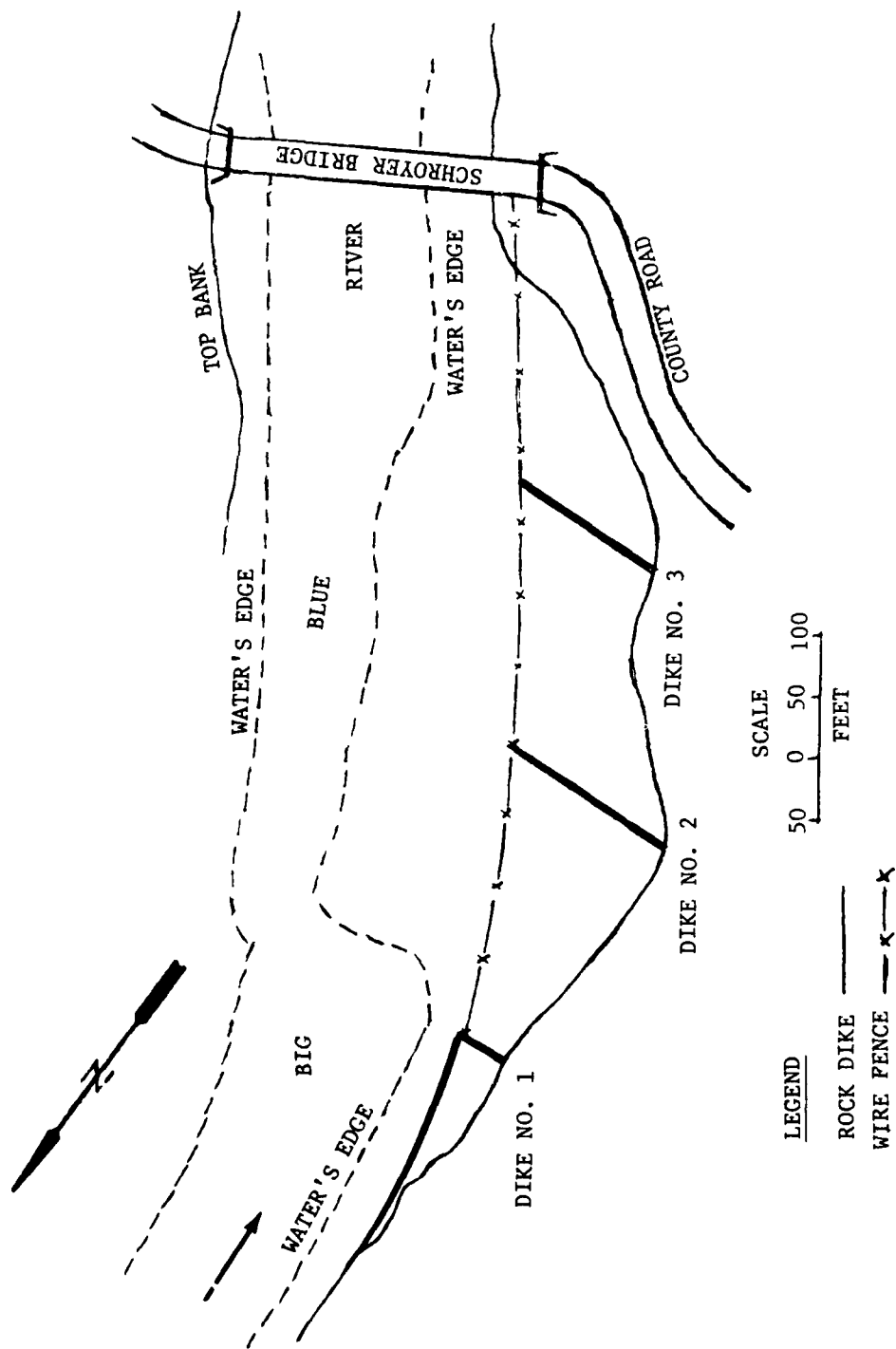


Figure 37. Big Blue River near Marysville, Kans. Plan view of wire-fence and rock-dike revetment



a. Downstream view of wire fence revetment. Schroyer Bridge is in the background



b. Wire fence revetment viewed toward end of dike 3

Figure 38. Big Blue River near Marysville, Kans. Two views of the completed project. (July 1977)



Figure 39. Big Blue River near Marysville, Kans. High flows in July 1978 undermined a section of the fence causing it to fail. This photograph was taken from the Schroyer Bridge during the 12 September 1978 WEC inspection visit.



Figure 40. Big Blue River near Marysville, Kans. Downstream view from the end of dike at the time of the WEC inspection visit (12 September 1978).

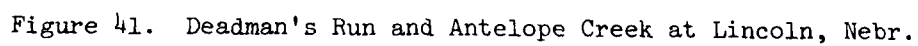


Figure 41. Deadman's Run and Antelope Creek at Lincoln, Nebr.

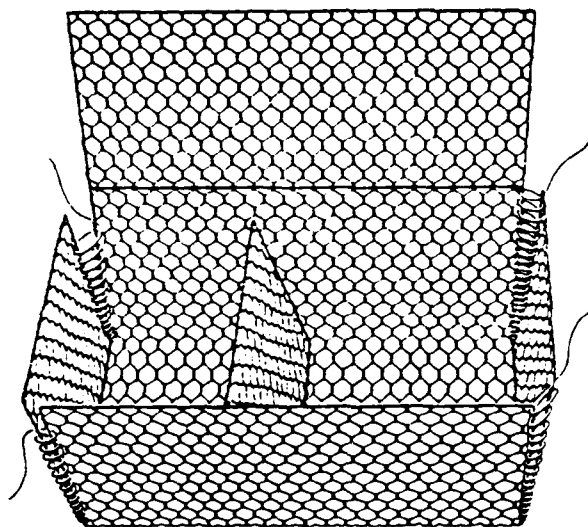
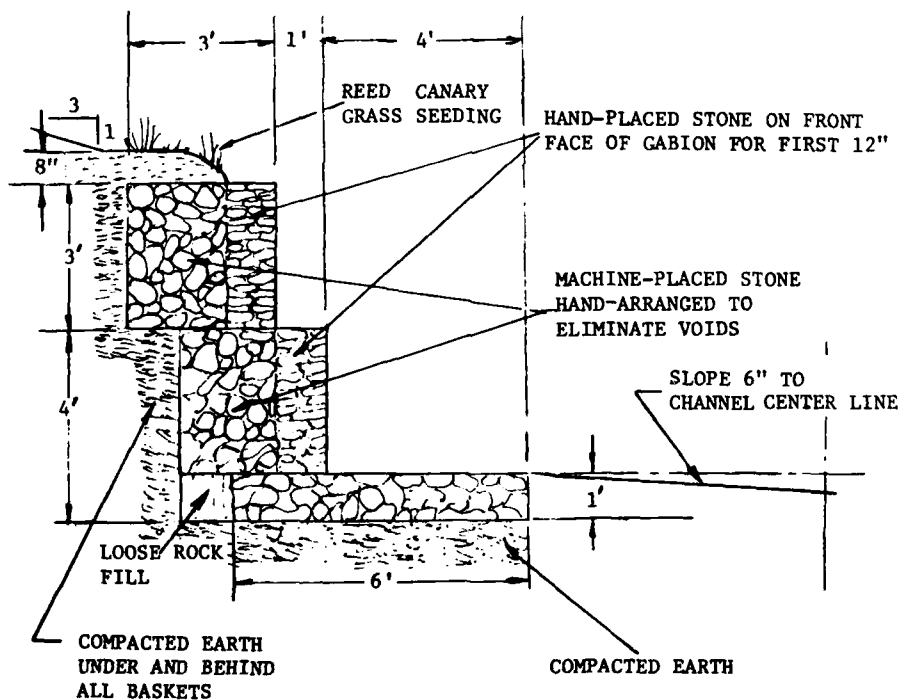


Figure 42. Typical gabion cage used for
revetment construction along Deadman's
Run and Antelope Creek



SCALE
3 2 1 0 3
FEET

Figure 43. Deadman's Run and Antelope Creek at Lincoln, Nebr.
Typical gabion revetment (adapted from file information pro-
vided by Clark Enerson Partners, Lincoln, Nebr.)



Figure 1. Large rectangular stone structure at Lincoln, Iowa. The stone is the front face of the wall and was covered in a thin layer of white plaster. This view was taken from the east end of the wall and looking westward from the Hillcrest Street bridge. (September 1977)



Figure 2. Large rectangular stone structure at Lincoln, Iowa. This view was taken from the west end of the wall and looking eastward. The stone is the front face of the wall and was covered in a thin layer of white plaster. This view was taken from the Hillcrest Street bridge. (September 1977)



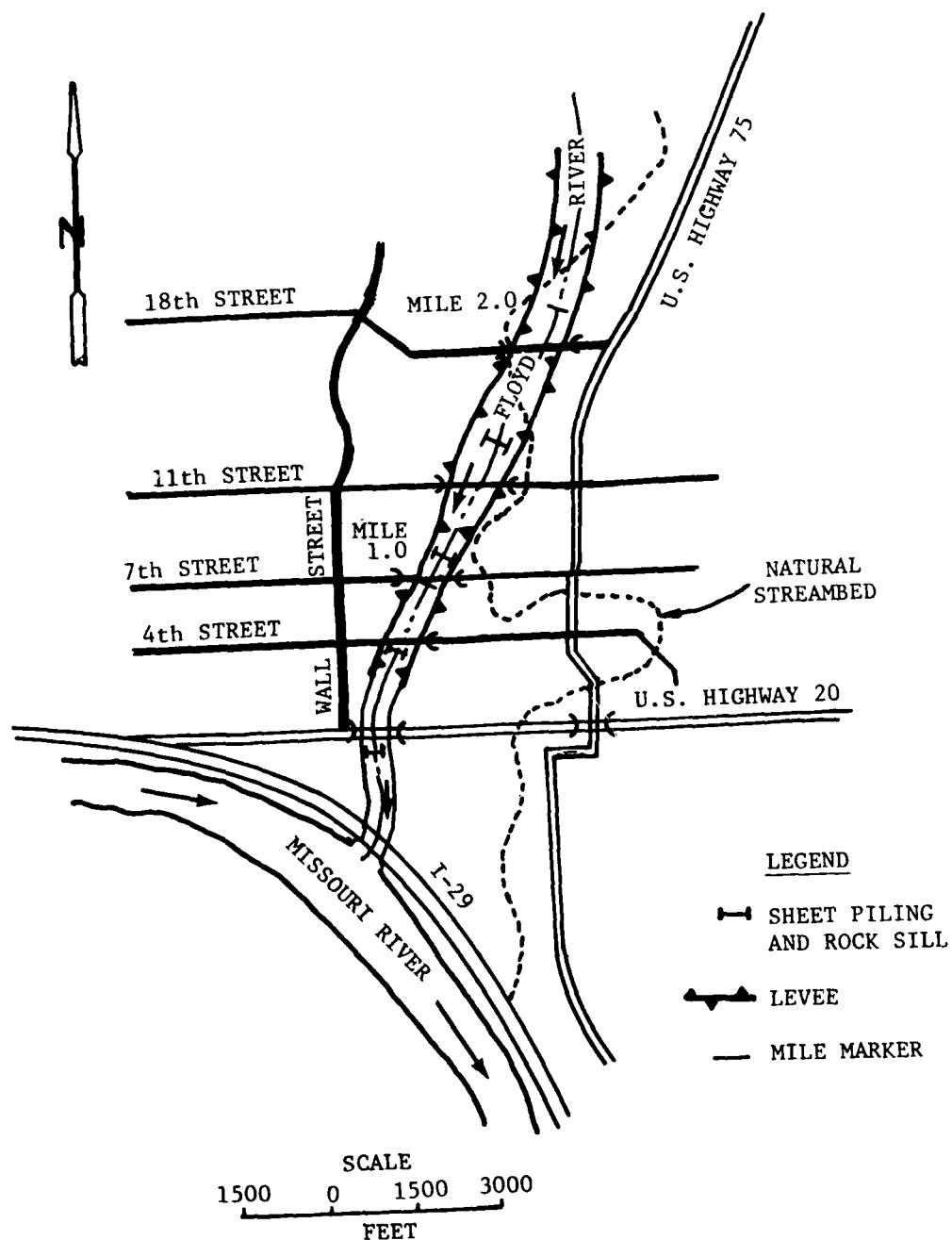


Figure 47. Floyd River at Sioux City, Iowa

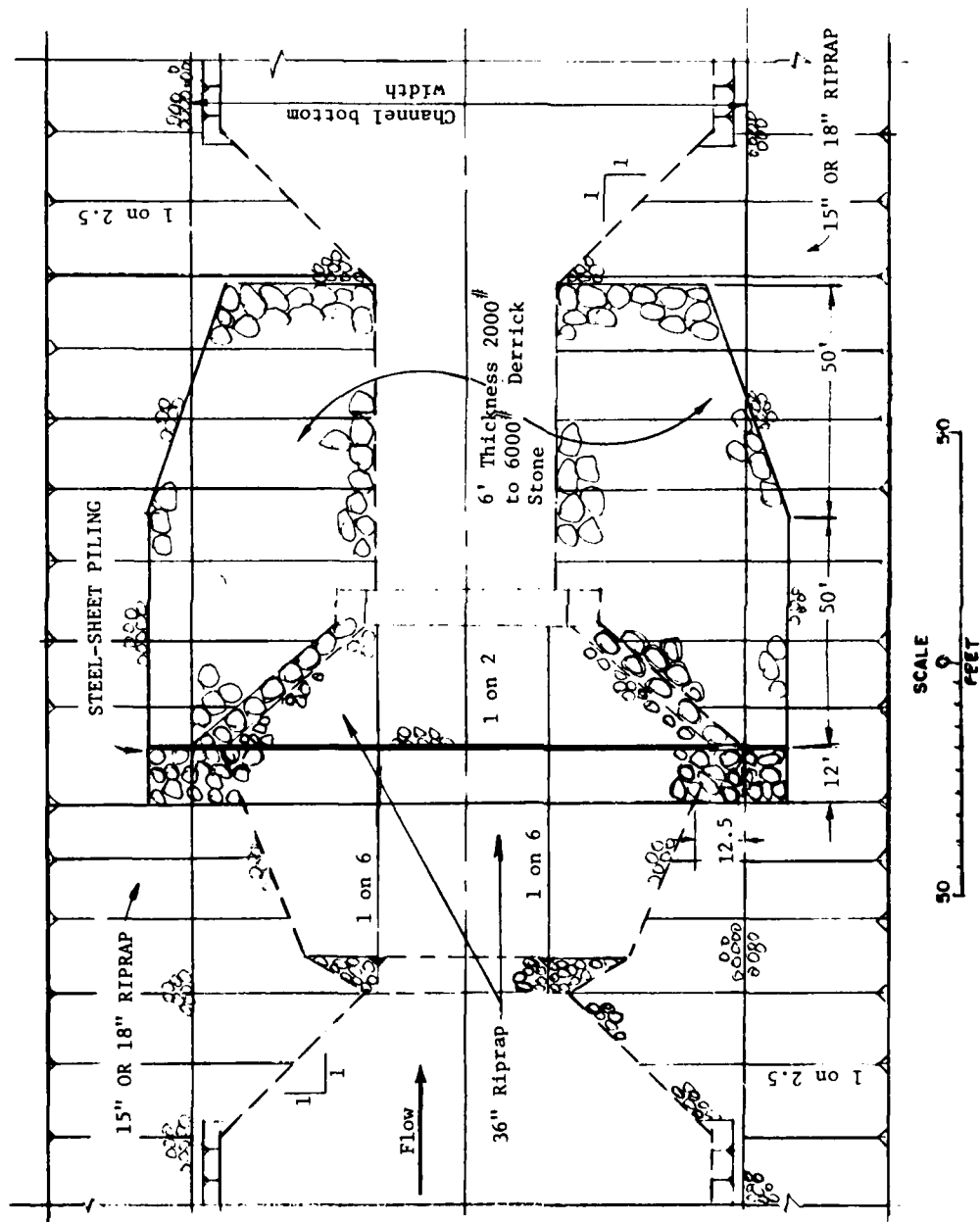
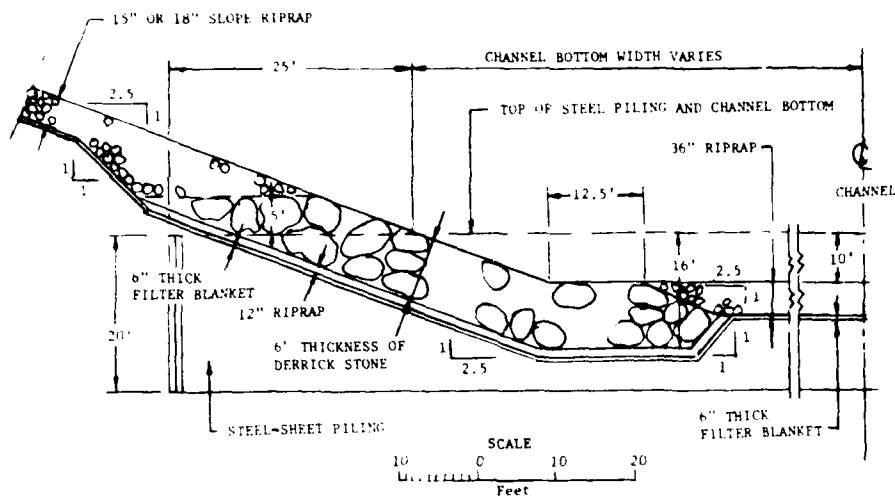
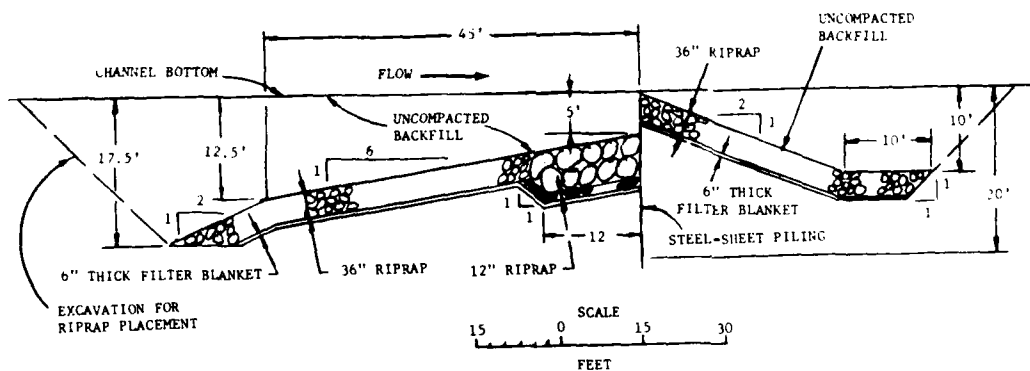


Figure 48. Floyd River at Sioux City, Iowa. Typical plan view of sheet piling and rock sill (Source: Reference 18)



a. Half-sectional view of sheet piling and rock sill



b. Center-line view of sheet piling and rock sill

Figure 49. Floyd River at Sioux City, Iowa. Half-sectional and center-line views of sheet piling and rock sill (Source: Reference 18)



Figure 10. Floyd River at Sioux City, Iowa. Distant view of the sheet piling and rock fill at mile 0.6 at the time of the WEF inspection. (South Street Bridge is in the background. 11 September 1978)



Figure 11. Floyd River at Sioux City, Iowa. Sheet piling and rock fill at mile 0.6. (20 September 1978)

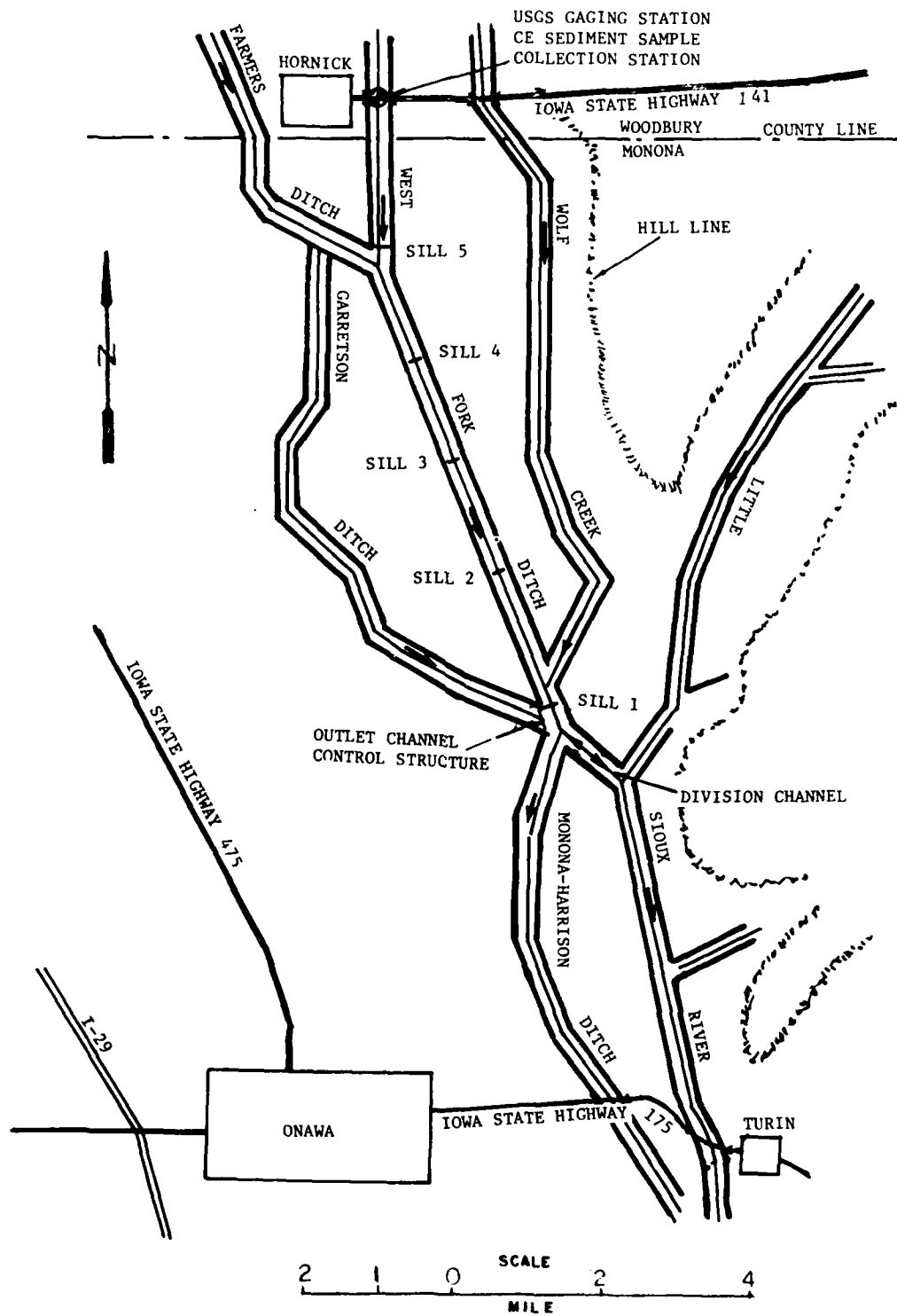


Figure 52. West Fork Ditch at Onawa, Iowa. Heavy lines paralleling streams represent levees

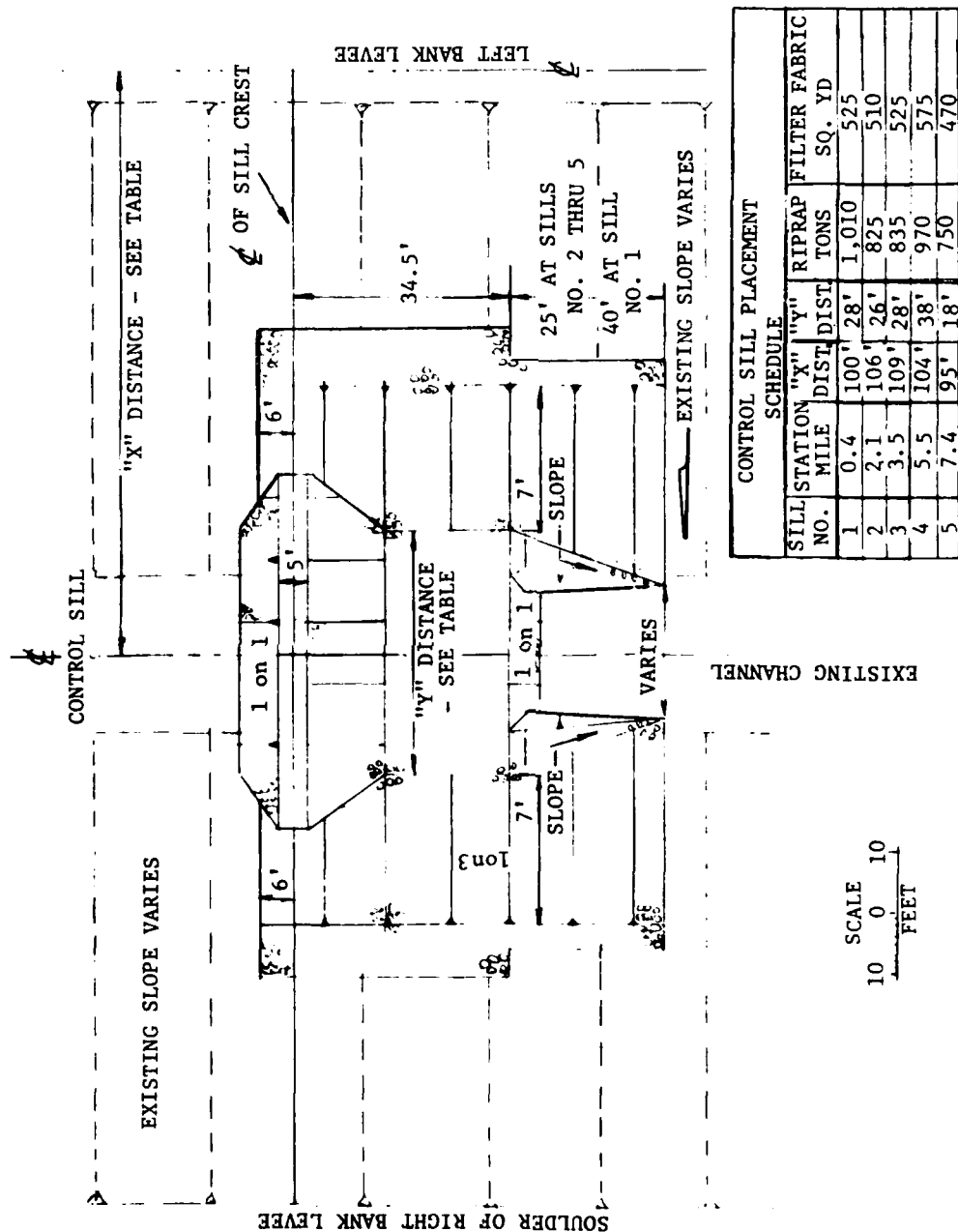


Figure 53. West Fork Ditch at Onawa, Iowa. Plan view of low-rock sills (file information provided by MRO)

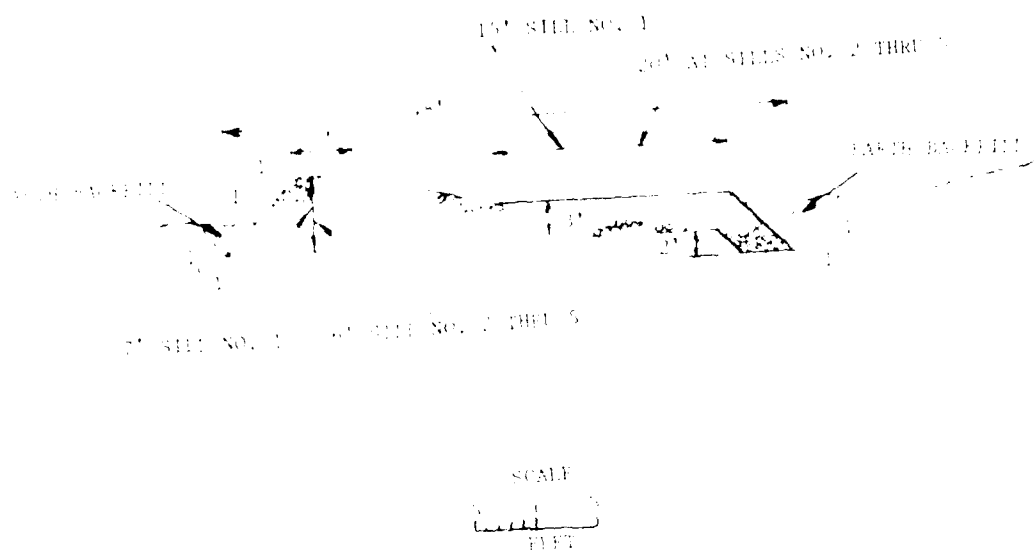


Figure 1. West bank of the river, showing the location of the dam and the location of the stills. The dam is located at the intersection of the river and the road. The stills are located at the intersection of the river and the road.



Figure 2. West bank of the river, showing the location of the dam and the location of the stills. The dam is located at the intersection of the river and the road. The stills are located at the intersection of the river and the road.

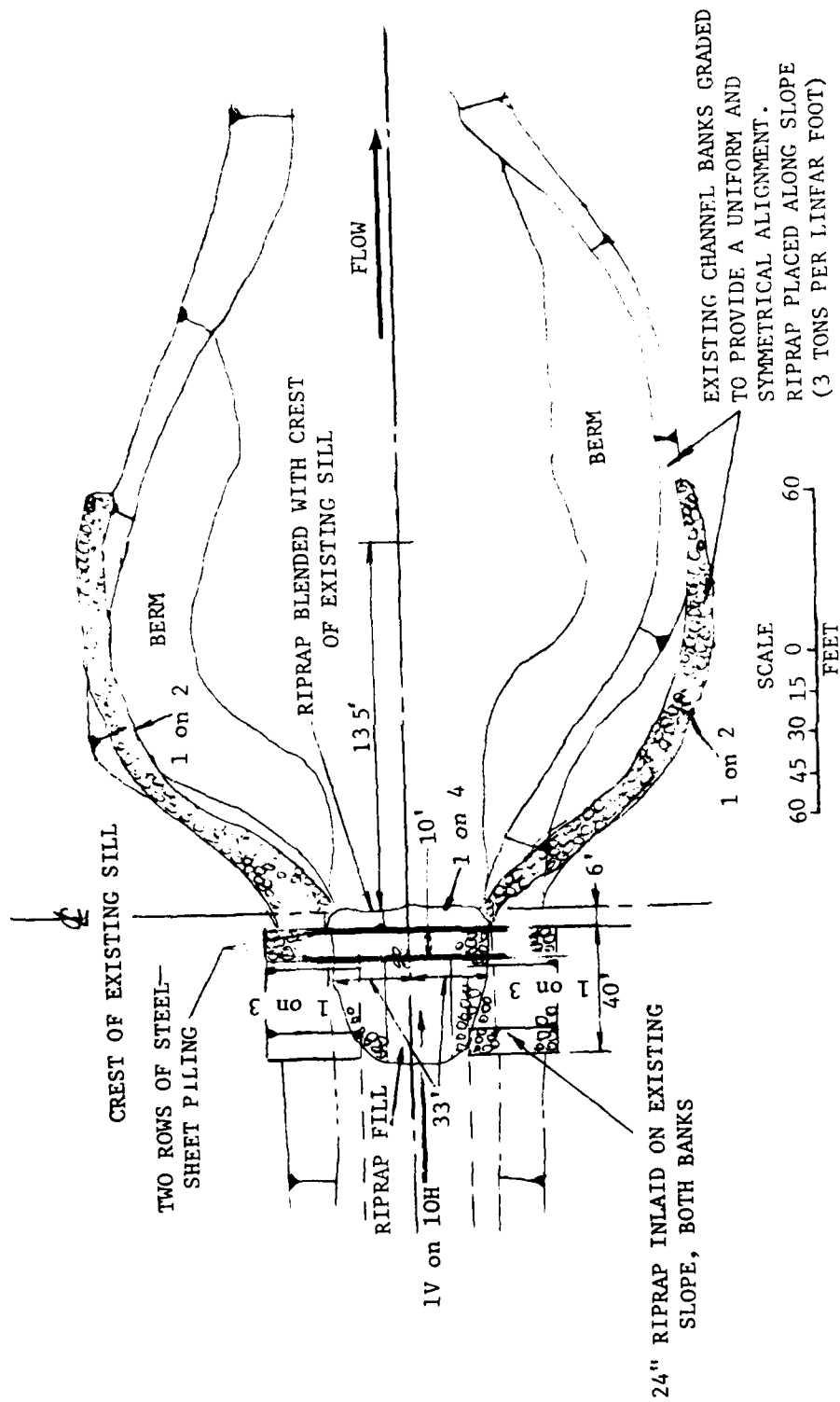


Figure 56. West Fork Ditch at Onawa, Iowa. Plan view of 1973 sill modification

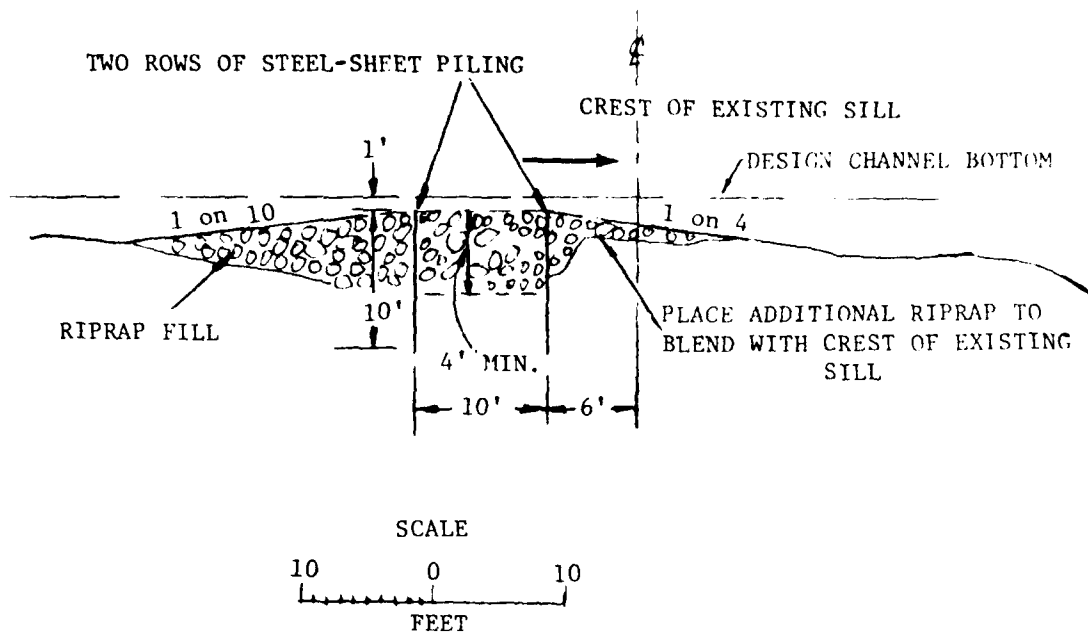


Figure 57. West Fork Ditch at Onawa, Iowa. Cross-sectional view (structure center line) of 1973 sill modification

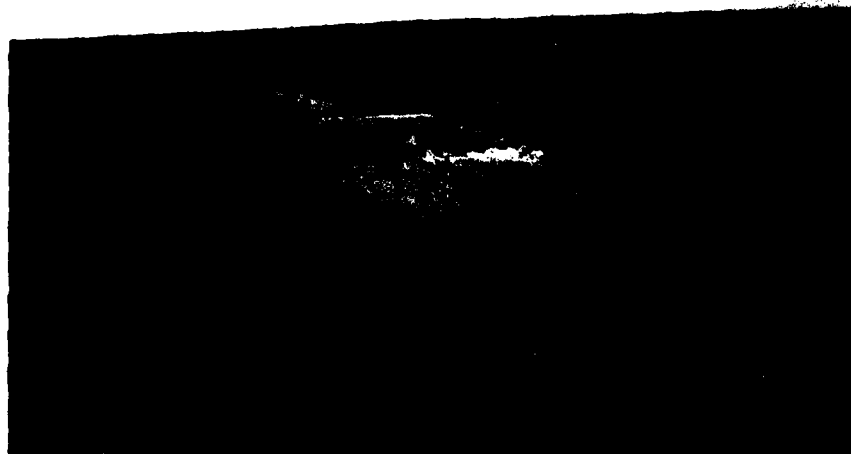


Figure 58. West Fork Ditch at Onawa, Iowa. Upstream view at sill 3 (20 September 1978)

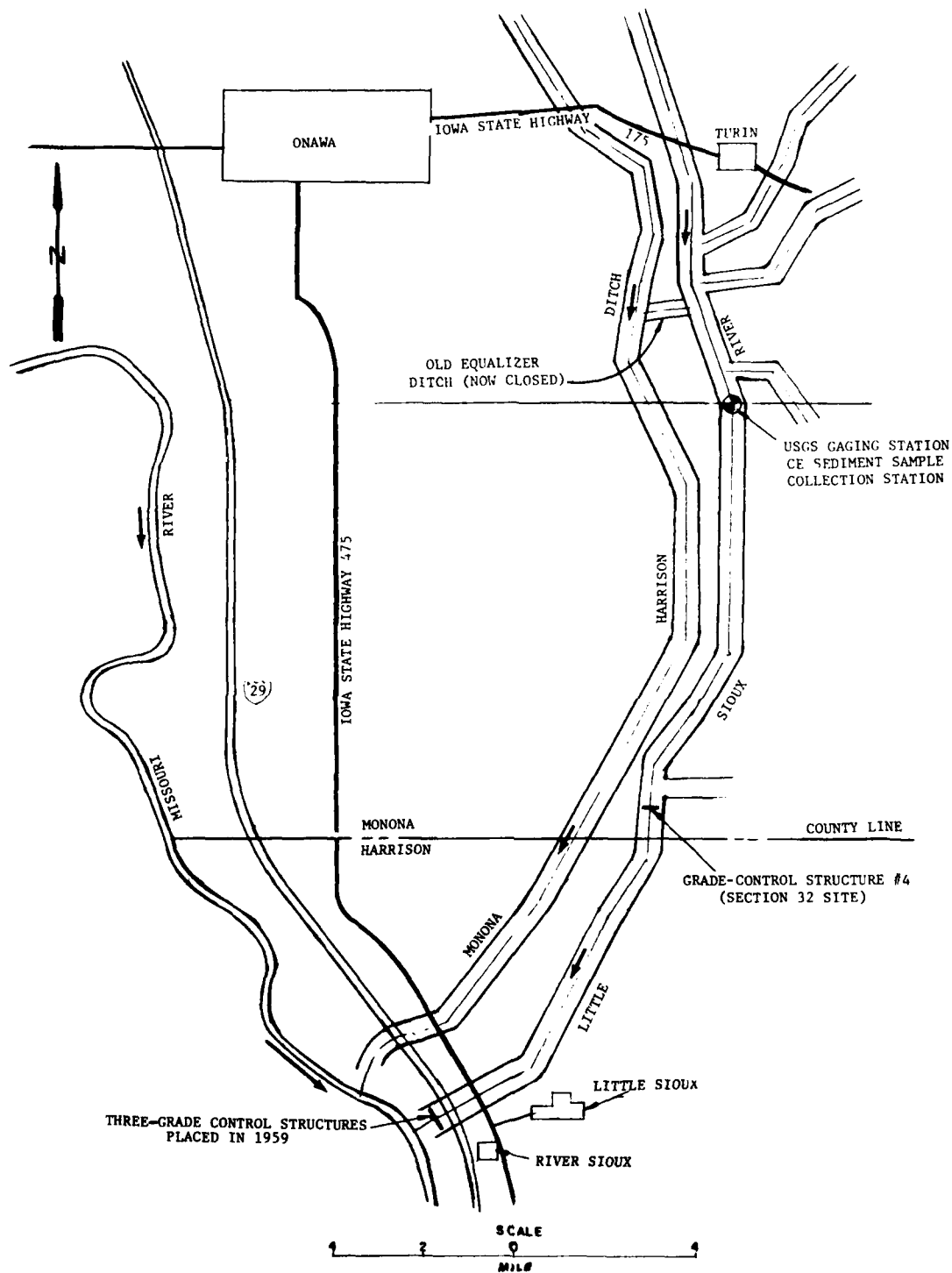


Figure 59. Little Sioux River at Onawa, Iowa. Heavy lines paralleling streams represent levees

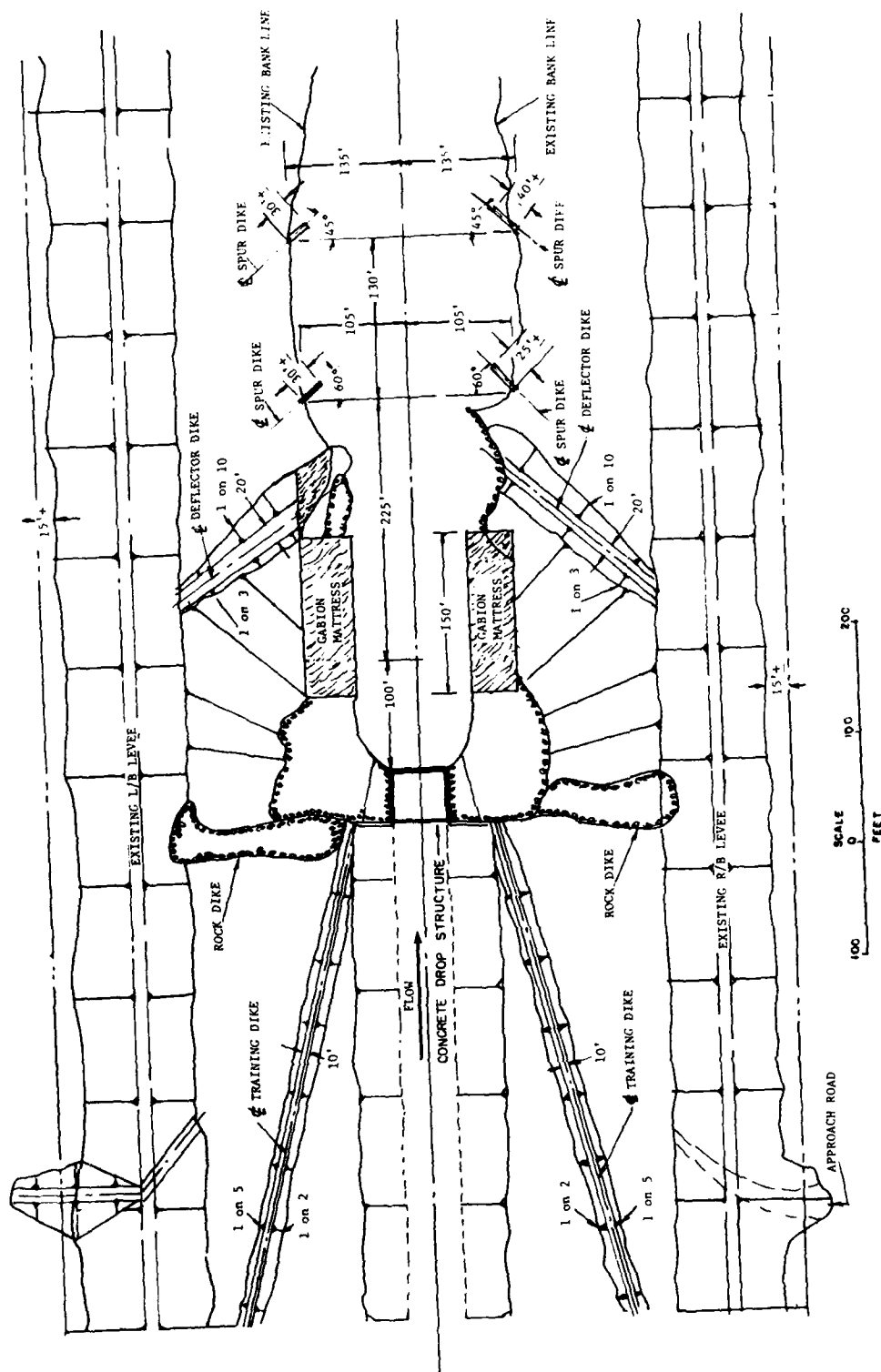


Figure 60. Little Sioux River at Onawa, Iowa. Plan view of grade-control structure #4 and gabion mattresses (adapted from WRD file information)



Figure 61. Little Sioux River at Onawa, Iowa. Damage to the stilling basin riprap as a result of high flows in April 1965 required the placement of grouted derrick stone to repair the basin



Figure 62. Little Sioux River at Onawa, Iowa. Toe failure of right-bank grouted stone section (20 September 1978)



Figure 63. Little Sioux River at Onawa, Iowa. Loss of grout on left-bank mattress (20 September 1978)



Figure 64. Little Sioux River at Onawa, Iowa. Part of the left-bank mattress failed under its own weight, due to scour under the mattress (20 September 1978)



Figure 65. Little Sioux River at Onawa, Iowa. Overbank return flows during March 1979 partially failed the right-bank mattress (July 1979)



Figure 66. Little Sioux River at Onawa, Iowa. Overbank return flows during March 1979 completely failed the left-bank mattress (July 1979)

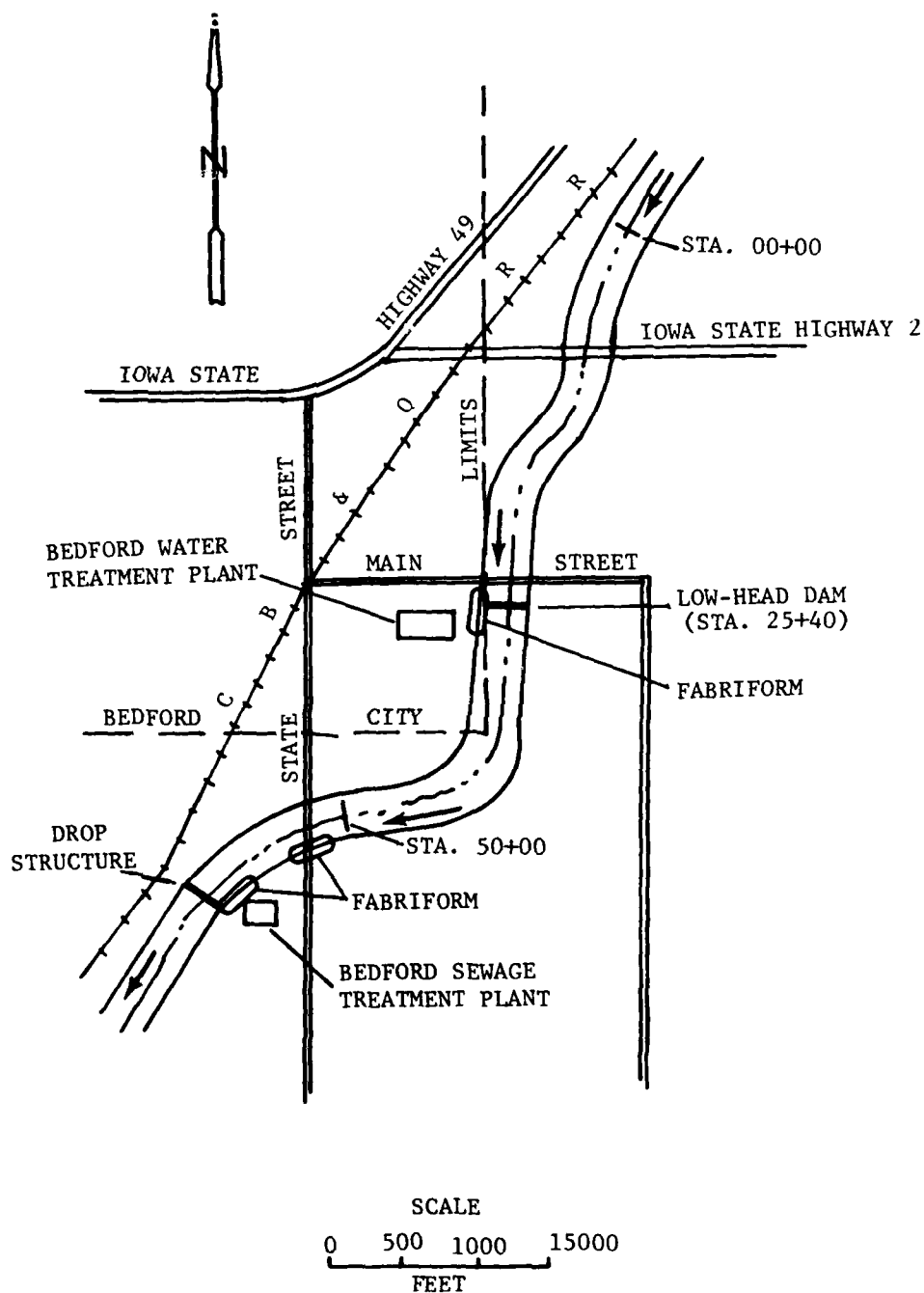


Figure 67. 102 River (East Fork) at Bedford, Iowa

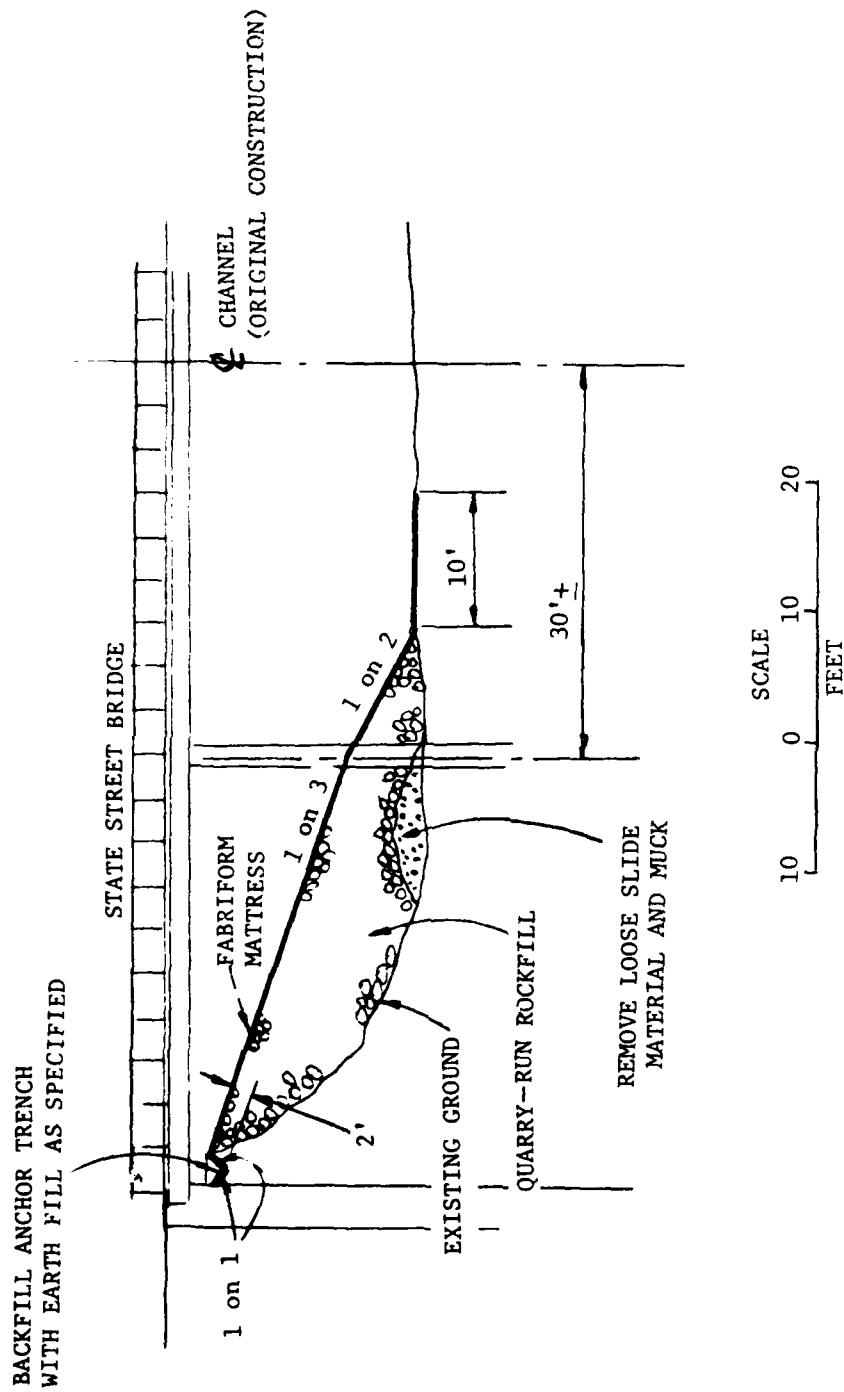


Figure 68. 102 River (East Fork) at Bedford, Iowa. Fabriform revetment on left bank at State Street Bridge (sta 52+42)

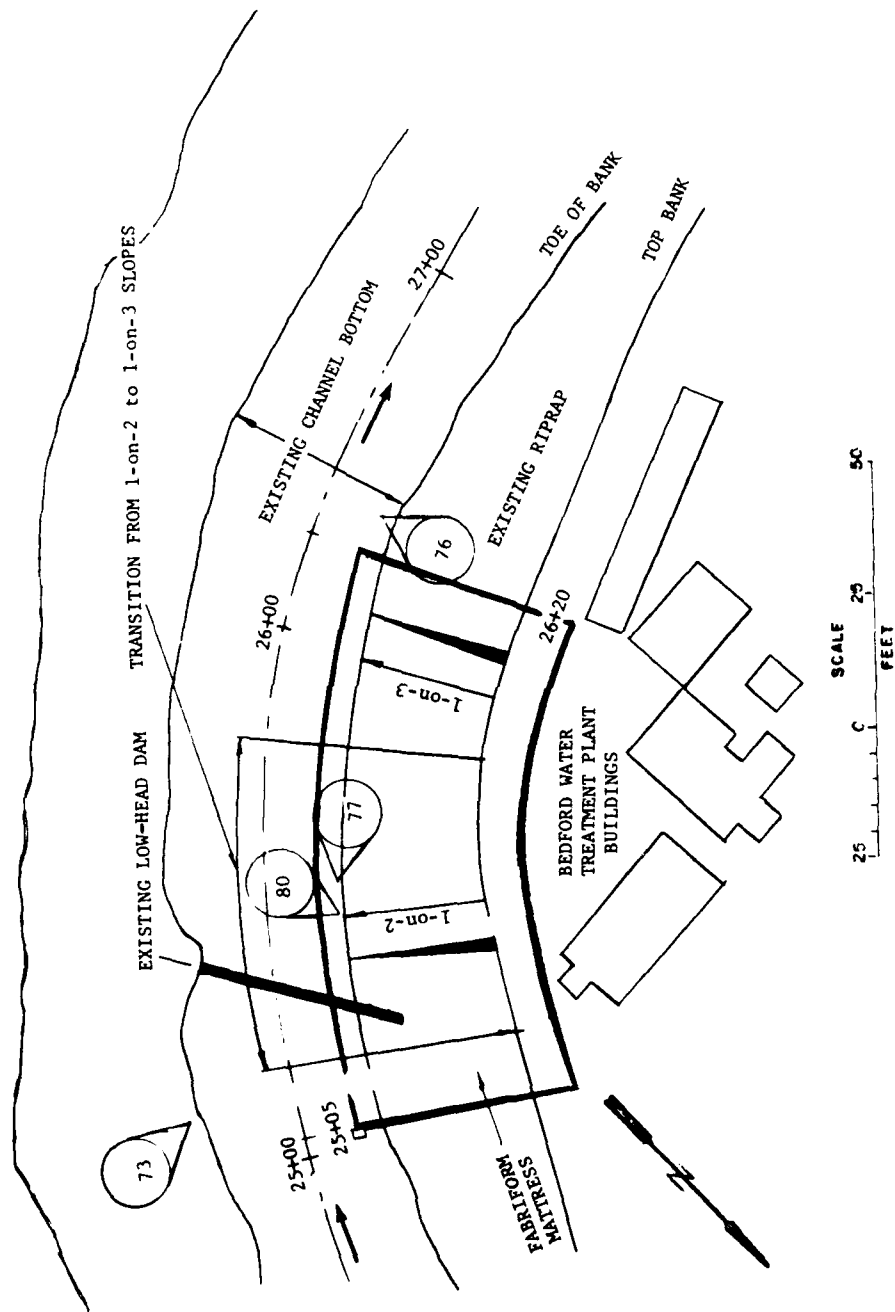



Figure 69. 102 River (East Fork) at Bedford, Iowa. Plan view of Fabriform revetment at Bedford Water Treatment Plant (sta 25+75). Orientation of field-of-view and location from which photograph was taken are represented by  for Figures 73, 76, 77, and 80

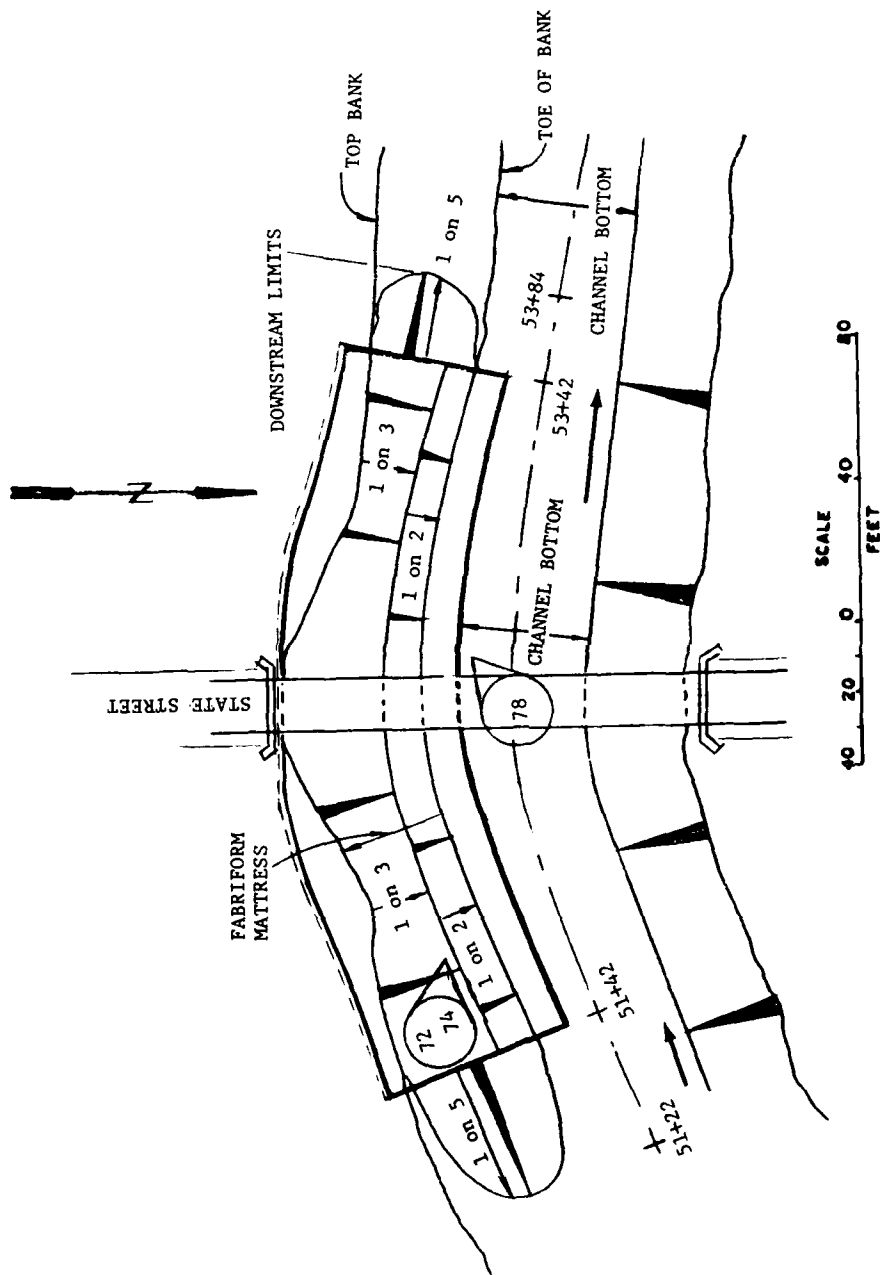



Figure 70. 102 River (East Fork) at Bedford, Iowa. Plan view of Fabriform re-
vetment of left bank at State Street Bridge (sta 52+42). Orientation of field-
of-view and location from which photograph was taken are represented by  for Figures 72, 74, and 78

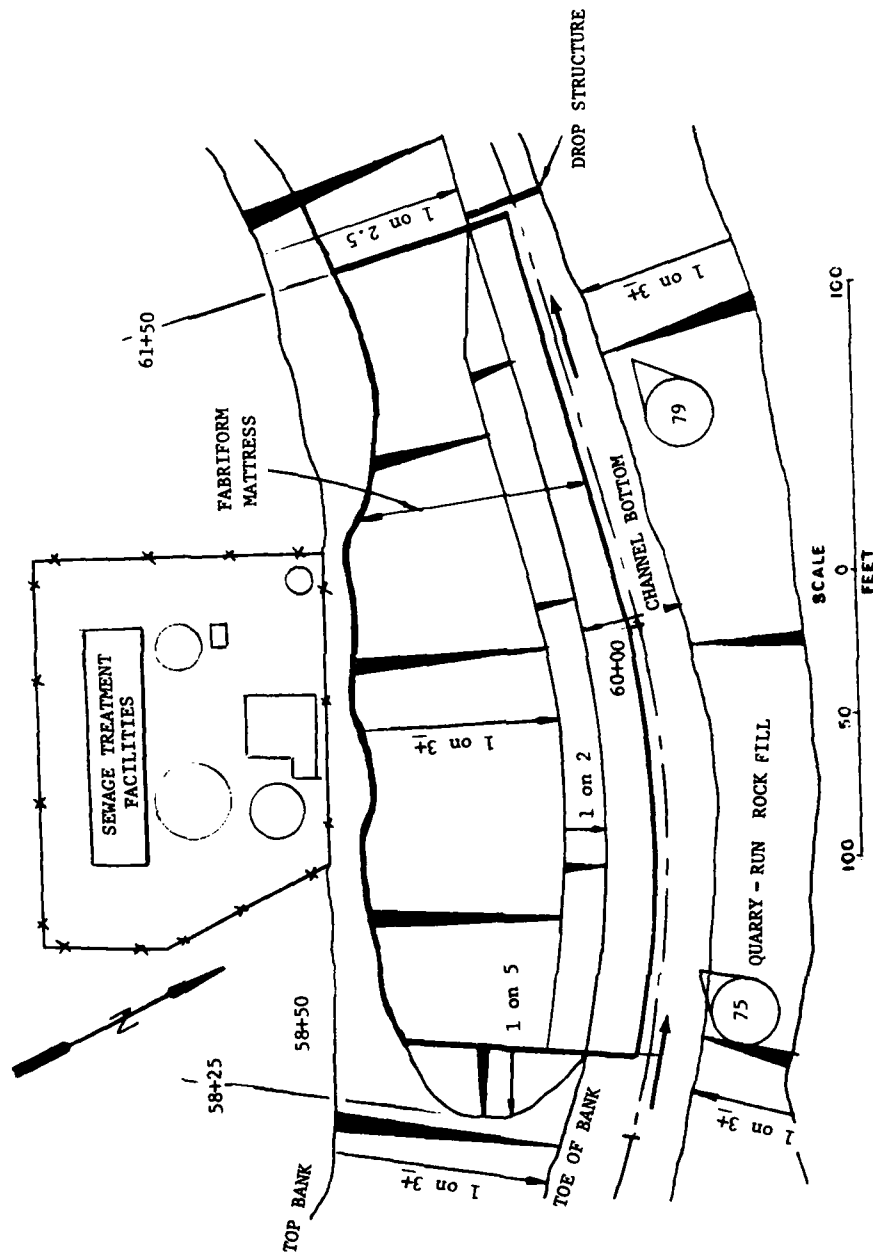



Figure 71. 102 River (East Fork) at Bedford, Iowa. Plan view of Fabriform revetment at Bedford Sewage Treatment Plant (sta 60+00). Orientation of field-of-view and location from which photograph was taken are represented by  for Figures 75 and 79



Figure 72. 102 River (East Fork) at Bedford, Iowa. Quarry-run riprap was used to shape the bank prior to placement of Fabriform. View is of the left bank of the State Street Bridge (photograph courtesy of Construction Techniques, Inc.)



Figure 73. 102 River (East Fork) at Bedford, Iowa. Completed Fabriform revetment at Bedford Water Treatment Plant (May 1974). Note low-head dam in foreground. (Photograph courtesy of Construction Techniques, Inc.)

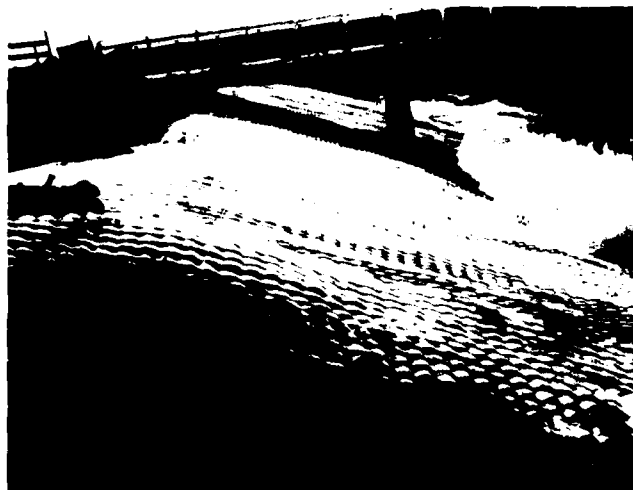


Figure 74. 102 River (East Fork) at Bedford, Iowa. Completed Fabriform revetment at State Street Bridge (May 1974). (Photograph courtesy of Construction Techniques, Inc.)



Figure 75. 102 River (East Fork) at Bedford, Iowa. Completed Fabriform revetment at Bedford Sewage Treatment Plant (May 1974). (Photograph courtesy of Construction Techniques, Inc.)

Figure 76. 10L River (East Fork) at Bedford, Iowa. High flows undercut the gabriiform mattress at several locations resulting in parts of the revetment breaking off and being swept into the center of the channel. (21 September 1973)

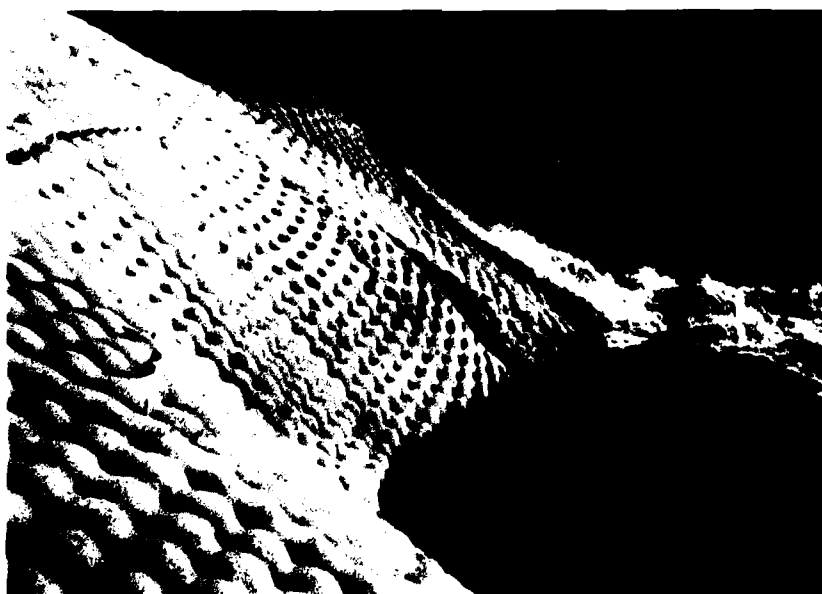


Figure 77. 10L River (East Fork) at Bedford, Iowa. High flows undercut the gabriiform mattress at the Bedford Water Treatment Plant resulting in subsidence of a section of the revetment. (21 September 1973)

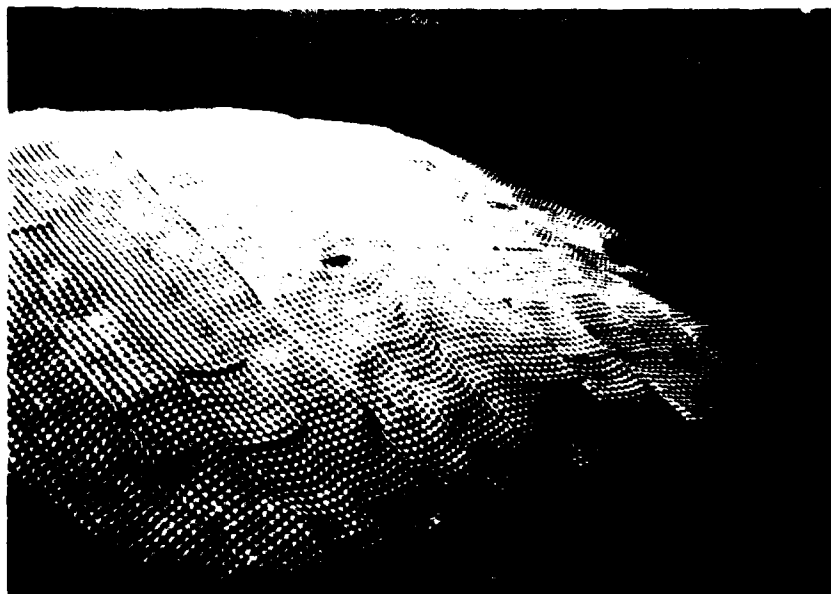


Figure 78. 102 River (East Fork) at Bedford, Iowa. At the time of the WES inspection visit, most of the Fabriform revetment was intact. This view shows the mattress downstream from the State Street Bridge (21 September 1978)



Figure 79. 102 River (East Fork) at Bedford, Iowa. Failure of drop structure at sta 61+60 (March 1979). Note damaged Fabriform mattress in left portion of view. (Photograph taken 22 January 1980)



Figure 80. 102 River (East Fork) at Bedford, Iowa. An MRK inspection of the Fabriform mattress at the Bedford Water Treatment Plant in March 1979 indicated that a large cavity had formed under the mattress, which collapsed (photograph taken 22 January 1980). The revetment was repaired with grouted rock and gabions in February 1980; the repaired revetment successfully withstood high flows during June of the same year. Note low-head dam in right portion of photograph

